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PREDICTING THE LOCATION AND DURATION OF TRANSIENT INDUCED LOW OR NEGATIVE PRESSURES WITHIN A LARGE WATER DISTRIBUTION SYSTEM

Richard C. Svindland
University of Kentucky

ABSTRACT OF THESIS

PREDICTING THE LOCATION AND DURATION OF TRANSIENT INDUCED LOW OR NEGATIVE PRESSURES WITHIN A LARGE WATER DISTRIBUTION SYSTEM

Surge modeling is a tool used by engineers and utility owners in determining the surge pressures or transients that may result from routine pump and valve operations. Recent surge modeling work has focused on low and/or negative pressures within water distribution systems and how those occurrences could lead to intrusions. Effective surge modeling is needed in order to determine if the intrusion potential exists and what mitigation is needed to prevent intrusions. This work focuses on the generally unexplored area of using surge models to predict the location and duration of transient induced low and/or negative pressures within large complex water distribution systems. The studied system serves 350,000 people in the southeast United States, has 65 MGD of pumping capacity at two treatment plants, over 1500 miles of main and 12 storage tanks. This work focuses on the correlation between field data and the surge model using the author's extensive operational knowledge of the system, access to real-time SCADA data, and different celerity or wave speed values. This work also traces the steps taken by the author to locate areas within the system that experienced transient induced low and / or negative pressure.

KEYWORDS: Surge, Water Hammer, Hydraulic Modeling, Transients, Negative Pressures

Richard C. Svindland, P.E.

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By

Richard C. Svindland, P.E.

Srinivasa Lingireddy, Ph.D., P.E.
Director of Thesis

Kamyar Mahboub, Ph.D.
Director of Graduate Studies

3-30-05

RULES FOR THE USE OF THESES

THESIS

Richard Cameron Svindland, P.E.

The Graduate School

University of Kentucky

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PREDICTING THE LOCATION AND DURATION OF TRANSIENT INDUCED LOW OR NEGATIVE PRESSURES WITHIN A LARGE WATER DISTRIBUTION SYSTEM

THESIS

A thesis submitted in partial fulfillment of the
requirements for the degree of Masters of Science
in Civil Engineering in the College of Engineering
at the University of Kentucky

By

Richard C. Svindland, P.E.

Lexington, Kentucky

Director: Dr. Srinivasa Lingireddy, Associate Professor of Civil Engineering

Lexington, Kentucky

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Dedicated to my wife and two sons

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CHAPTER 1
INTRODUCTION

Computer models have been used extensively over the past two decades to model flows and pressures within a water distribution system. More recently hydraulic models have been used to model water quality and water hammer, surge or transient events within water distribution systems. In the American Water Works Research Foundation (AWWARF) Project No. 2686 Report "Verification and Control of Pressure Transients and Intrusion in Distribution Systems" conclusions were drawn regarding System number 2 (System #2) that the model results did not correlate well with field observations. System #2 serves 350,000 people in the southeast United States, has 65 MGD of pumping capacity at two treatment plants, over 1500 miles of main and 12 storage tanks. This work further investigates the correlation between field and model results using the author's extensive operational knowledge of System #2, access to real-time SCADA data, and access to boundary conditions, all of which were not considered adequately in the previous study.

This work will discuss current regulations in regards to minimum pressure requirements and cross connection programs and will present some calculations as to what potential flow could revert back into the distribution system under low or negative pressure conditions. This work will also provide tips to be used by engineers and water distribution system operators to locate areas within a distribution system that may potentially experience low or negative pressure and what precautions should be followed.

The phenomenon known as water hammer, surge or transients is well documented and has been known to exist for over a 100 years. The equation developed by Joukowsky in 1898 is widely used today even though its derivation was conducted so long ago. In 1976, a Scottish research student named Alexander Anderson had a paper published in the Journal of the Hydraulics Division regarding an Italian engineer named L. F. Menabrea who published a short but concise paper in 1858 regarding water hammer and Jules Michaud who is frequently attributed to the earliest water hammer analysis in 1878. (Anderson, 1976)

Many textbooks today still use the fundamental equations developed by Joukowsky and others. E. Benjamin Wylie, widely considered to be an international expert on water hammer published an article in 1984 in the Journal of Hydraulic Engineering titled “Fundamental Equations of Water hammer” with discussions by leading hydraulic experts from around the world including Dr. Samuel Martin of Georgia Institute of Technology, whom taught the author of this paper’s undergraduate hydraulics class while he was at Georgia Tech in the late 1980’s. In Wylie’s paper the basic continuity equation was discussed, as well as how to handle the slope of the pipeline in solving the equations. (Wylie, 1984)

As mentioned earlier, the phenomenon known as water hammer has been known for over 100 years. The difficulty with water hammer, surge or transient analysis is how to solve the equations to give a complete account of the surge wave as it passes through a point.

As an active practicing professional civil engineer, the author has routinely used the Joukowsky equation coupled with an instantaneous valve closure time to quickly calculate the maximum expected surge pressure or head that could be produced. The author would also consider rapid pipe draining as the governing case for low pressure, which could lead to pipe wall buckling. As luck would have it, on most of the systems designed by the author, the magnitude of the surge pressure produced was less than the strength of the pipe and the prevention of pipe buckling due to rapid drainage could be easily handled with air / vacuum valves. This meant that nothing else “needed to be done” and the effects of high and low pressures as a result of a velocity change were fairly easily handled.

This is not to say that there were not times where surge problems needed devices in addition to pipe strength and air / vacuum valves because there were. The author has been involved in several pipeline designs where hydro-pneumatic surge vessels were used to eliminate secondary surge waves produced by water column separation and the resulting collapsing of these air pockets. The point to this discussion is to say that low and high pressure waves that occur within a water distribution system can be handled rather easily but there are other aspects of low pressure surge waves such as pathogen intrusion that warrant further study.

The main purpose of the AWWARF Project No. 2686 was to test the assumption that low pressure transients can result in contamination to the water distribution system when pressures of the water surrounding the water main are greater than the internal pressure (LeChevallier, et al, 2002). The water surrounding the water main may be there due to either a water main leak, surcharged sanitary sewer, or high ground water levels. AWWARF Project 2686 was commissioned to investigate several items, one was the use of high speed data recorders to try and capture the low and / or negative pressure surge waves, the second was the use of conventional pressures recorders and the third item was to investigate the applicability of computer surge models such as Surge2000 to predict surge waves.

AWWARF Project 2686 was a success in that it demonstrated that low and / or negative surge waves do exist and can be captured with high speed data recorders and in some cases traditional pen and chart recorders. AWWARF Project 2686 was not successful in correlating the field data with computer surge models. This work improves upon the modeling results. The benefit of having a computer model that can predict with some level of accuracy the location and duration of low and / or negative pressures is that it gives water distribution system operators and engineers a powerful analysis tool that can be used to implement cost effective solutions.

1.1 State of Hydraulic Modeling

Hydraulic modeling of water distribution systems is common. Many water utilities use hydraulic models routinely to assist in the design of system expansions and improvements such as new mains to serve new developments and new mains to replace aging infrastructure. Because hydraulic models rely heavily on field data to be able to simulate actual field conditions they lend themselves well to being used as a tool to locate areas of high friction loss (i.e. low Hazen Williams C-values). Hydraulic models are also used during comprehensive planning studies to assist in the preliminary design of a water system. Because hydraulic models can run on personal computers (PC) they have become easier to use and give design engineers the ability to run many alternatives quickly.

There are currently several firms, companies or institutions that have created and/or are marketing hydraulic modeling software. The most common in use today within the United States are Pipe2000 by the Civil Engineering Software Center at the University of Kentucky, WaterCAD by Haestad Methods, Inc., H20Net by MWH Soft and EPANET by the Environmental Protection Agency. Other software products were also developed by Stoner and Pitometer Associates and have also been used to model water distribution systems. Each of these products has evolved into user-friendly platforms that can run as stand alone platforms or within other programs such as AutoCAD (a widely used drawing software by AutoDesk). Each of the products is also similar in that they compute flows in pipes and pressures at nodes (junctions) by solving looped headloss equations simultaneously. The programs differ in the methodology used to solve these equations. H20Net and WaterCAD use an EPANET engine to solve for the flows and pressures. EPANET solves for the pressures at the nodes first and then computes the flows in the pipes to achieve the previously computed pressures. In contrast, Pipe2000 computes the flows in the pipe first then computes the resulting pressures at each node.

This work will not focus on which methodology is best or more stable under different conditions. The above items are mentioned as information only and do not affect this work other than to say, for the hydraulic modeling performed for this paper, the Pipe2000 product which solves for the pipe flow first and then the pressures at the nodes was used. It is unknown and beyond the scope of this work to determine if the hydraulic modeling results would be different if another software product was utilized.

1.2 State of Surge Modeling

Like hydraulic modeling software, surge-modeling software has also increased in usage since the increased use of PCs in the workplace. It was not that long ago that most surge modeling software was solely created and used by the academic community on a project-by-project basis. A colleague once noted that “for surge models, it took PhD’s to understand how to input the data and how to understand the output.” The main reason for this was due to the complex nature of solving the equations. Most of these models used the “method of characteristic” (MOC) approach to solving complex equations.

In 1966, Dr. Don C. Wood, et al, first introduced the concept that became known as the “wave plan method” (WPM) analysis approach to solving unsteady flow in closed conduits. Over the years, this work evolved into the creation of the SurgeX.X (X.X referring to version) software created and marketed by the Civil Engineering Software Center at the University of Kentucky. More recently, the developers of WaterCAD and H20Net have also started marketing surge modeling software. Haestad Methods markets a surge modeling product called HAMMER that is based on a MOC engine developed by the Environmental Hydraulics Group. MWSoft markets a surge modeling product called H20Surge / InfoSurge that is based upon a “wave characteristic method” (WCM). WCM is a hybrid between the WPM and MOC and based on information on the MWSoft website (www.mwsoft.com) is the “fastest, most efficient, most rigorous and stable algorithm for solving hydraulic transients.”

This work will not focus on which methodology is fastest, best or more stable under different conditions. The above items are mentioned as information only and do not affect this work other than to say that the surge modeling performed for this paper uses the Surge2000 product, which uses WPM. It is unknown and beyond the scope of this work to determine if the surge modeling results would be different if another software product or solving methodology was utilized.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction to Literature Review

As previously discussed, surge, transients or water hammer has been well documented over the last 100 years. As part of the literature review, databases for the American Society of Civil Engineers, which includes the Journal of Hydraulics (both Division and Engineering) as well as its predecessor the Journal of Transportation Engineer and the American Water Works Association were searched for articles on water hammer, surge or transients. Many papers were found, such as paper by George Belonogoff titled “Computer Simulation of Waterhammer Effects” and a paper by Duncan McInnis and Bryan W Karney titled “Transient in Distribution Networks: Field Tests and Demand Models” but none of the papers dealt with predicting the locations and magnitudes of low pressure transients using computer models in large water distribution systems.

The Belonogoff paper, which was published in 1972 used a Fortran based MOC solver to compute transients at a large nuclear power plant. Several papers will be discussed and compared to this work. The first is a paper based on low-pressure problems in a large water distribution system in Austin, Texas. The second paper is based on field monitoring and surge modeling in a large water distribution system in Davenport, Iowa. The third paper, the previously mentioned McInnis paper, discusses surge modeling work and computer modeling done in a large distribution system. The fourth paper will briefly discuss the effect of skeletonization on transient model results. A discussion of papers regarding the link between health effects and transients will be discussed.

2.2 Transient Induced Low Pressures in Austin, Texas

The December 1994 issue of the American Water Works Association Journal features a paper by Thomas M. Walski and Tersa L. Lutes titled “Hydraulic Transients Cause Low-Pressure Problems” and is about “mysterious short-term pressure drops at the top of Cat Mountain.” Cat Mountain is located in an area of high elevation and is a part of the Austin, Texas water distribution system. The Cat Mountain area is located in Northwest A pressure zone that has an average daily demand of 12.5 MGD with an additional flow of 5.8 MGD passing through the zone to another service territory.

As a comparison, the City of Austin's water distribution system serves approximately one half million people at elevations of between 750 and 1050 feet. This makes the Austin system slightly larger than System #2 of this work, which serves approximately 350,000 people. The elevation of the Austin system is about the same as the elevation of System #2 in that System #2 varies between 850 and 1180. Even though the Austin water system is larger than System #2, the Northwest A pressure zone is smaller than System #2 since System #2 has average flows of 42 MGD. The pumps used to pump into the Northwest A pressure zone varied in size, but generally an 8000-gpm pump was utilized. This is approximately the same size pump used during the drawdown tests at WTP1 in System #2.

The Austin paper discusses the analysis undertaken to determine the culprit for the low-pressure problems. Items discussed include large demands, faulty air / vacuum valves, faulty pressure reducing valves, poor carrying capacity, water theft and operating procedures. Similar to the fieldwork performed for this work, resulting surge pressures below 20 psi were recorded in Northwest A pressure zone. No negative values were recorded. The lowest recorded pressure value was 9 psi. The Austin paper does not discuss what analysis tools were utilized during the study other than to mention that a celerity or wave speed of 3000 fps was utilized. This value is consistent with values used in this work. In summary, the paper concluded that the isolation of storage tanks coupled with routine pump shutdowns was the culprit for the observed transient induced low pressures. The Austin paper recommended that storage tanks not be isolated from the distribution system during pump shutdowns, because the tanks help to dampen the surge wave. Their field data showed and confirmed this assumption and their assumption is consistent with textbook discussions regarding the difference between a tank and a dead end main.

Unlike the Austin system that changed its operation to keep storage tanks online, the System #2 distribution system does not have the ability to keep tanks online. This is due to the fact that most of the tanks in System #2 are pump-storage tanks and that only a couple of tanks located far away from the treatment plants are online floating tanks which could help dampen surges. One would expect, based on the Austin paper that surge waves in System #2 would have a greater effect due to the limited amount of storage tanks. Based on this work, that assumption appears to be correct.

2.3 Low Pressure Monitoring and Modeling in Davenport, Iowa

Richard W. Gullick, Mark W. LeChevallier, James Case, Don J. Wood, James E. Funk and Melinda J. Friedman authored a paper in March 2004, tilted "Application of Pressure Monitoring and Modeling to Detect Low Pressure Events in Distribution Systems". This paper was submitted to AQUA for publication and like the AWWARF Project 2686 was a project funded by EPA and AWWARF to determine if contaminants could enter a water distribution system due to low pressure transients. This paper presented the results of over 1.4 years of data logging within the water distribution system of Davenport, Iowa to determine if low and / or negative pressures occurred. Over the 1.4 years nine occasions of pressures below 20 psi were observed and no pressures below 0 psi were recorded. This in contrast to this work where eleven occurrences of pressures below 0 psi were recorded within System #2.

Although the Davenport study is similar to this work there are several differences that separate the two works and allow this work to further the study into predicting low and / or negative pressures within large water distribution systems. The differences are:

- The Davenport system and model are considerably smaller than that of System #2. The Davenport water system serves 45,800 customers vs. over 105,000 customers for System #2.
- The Davenport system consists of approximately 541 miles of main versus over 1500 miles of main for System #2.

- The Davenport surge model consisted of 1703 pipes, 1146 nodes, 12 supplies, 30 pumps and mains between 6 and 24-inches vs. 2516 pipes, 1758 nodes, 2 WTP, 12 tanks and mains between 2 and 36-inches for the System #2 model.
- The Davenport paper only modeled surge waves using surge wave speeds of 3000 fps vs. this work that utilized six different values of wave speed for each modeling scenario.
- As part of the Davenport fieldwork, no treatment plant power outages occurred which would have caused a greater low pressure event. The surge modeling work was thus focused around determining what impact would be expected to occur if power was lost at the WTP. This in comparison to this work where a lightning strike took out all the running high service pumps at WTP 1 (scenario #3).
- The total amount of time in which all nodes were less than 0 psi was determined based on surge model runs for expected power failure scenarios. These values were not tested against field data since no actual pressures below 0 psi were recorded. This is in contrast to this work where actual values below 0 psi were recorded in the field and compared to this work.

Despite the difference between the Davenport paper and this work, there are several similar items that should be mentioned and discussed to indicate how surge models can be used as tools to the engineering and water distribution operations groups.

- Both Davenport and this work use the pressure contour generation tool within Surge2000 to show the locations of low and / or negative pressures.
- Both Davenport and this work used and had good results with a 3000 fps wave speed.

- Both Davenport and this work confirmed that routine pump shutdown operations could cause pressures below 20 psi to occur.

The work under taken by the AWWARF Project 2686 was the next step based upon the Davenport work. The drawback to the AWWARF project 2686 was that the team was unable to get the surge models to correlate well with the filed data. This work, as mentioned previously was done to improve upon the work started by the Project 2686 team and thus ultimately build on the work started in Davenport.

2.4 Transient Field Test and Demand Models in Calgary, Canada

In 1995, Duncan McInnis and Bryan W. Karney published a paper in the Journal of Hydraulics titled “Transient in Distribution Networks: Field Tests and Demand Models”. This paper was based on surge modeling work within the City of Calgary’s water distribution system (Calgary, Canada). The reason for the paper was to explore “the relatively unexplored area of transients in complex pipe networks.” The paper explains that one of the reasons for lack of work in the area of transient studies in complex distribution systems was because it was widely believed that complex distribution systems, by their very nature, contributed to the rapid dampening of surge events. The paper explains that there is little rational for that thought and that they had found it to be contrary to some of their previous work.

One item that the Calgary paper addressed was the allocation of demands within the complex distribution system. The Calgary paper explored three methods of handling these demands. The three methods were discrete demands at nodes, orifice based demands and at nodes and distributed demands along the pipes. As a comparison, for the surge model done as part of this work, orifice demands at nodes are utilized.

The Calgary distribution system was modeled using a proprietary software called TRANSAM, which utilizes a MOC engine to solve for pressures at nodes and was compared to field data obtained form actual pump shutdown operations.

In review of the results, the field data and the surge model data correlated extremely well at several monitoring sites. The surge model created for this work; however, was not extremely complex. The model contained 132 pipes and 123 nodes, three pump stations and one reservoir with overall length of pipe of around 90 kilometers or 55 miles and had an average demand of 2.1 MGD. The waves speed utilized varied between 1150 meter / second and 1200 meter / second or 3772 fps and 3937 fps. As previously indicated the size of the Calgary model is approximately 10 smaller in terms of length monitored and 20 times smaller in terms of average demand than the model created for System #2.

The purpose of the Calgary work was not to determine and locate low and /or negative transient, but to show that complex water distribution systems need to be studied further because transients due exist. As part of the conclusion, the author's state that their work is only the beginning in trying to understand transient behavior in complex water distribution systems. and that additional field testing and modeling will be needed. This work is a logical next step in that it seeks to correlate surge model results with real life field data.

2.5 Effects of Skeletonization on Surge Models

Thomas W. Walski, Jean-Luc Daviau and Samuel Coran wrote a paper titled "Effect of Skeletonization on Transient Analysis Results". No date was indicated on the paper except a reference to the year 2003 was found within text body so the paper is no older than 2003. The purpose of the paper was to create surge models that represented a real water system, then run different skeletonizing routines to reduce the number of pipes within the model, re-run the surge analysis and determine the effects on the final results.

The model created for this skeletonizing work contained 261 pipes covering 18 miles of length with pipe sizes between 4 and 12-inches. The model contained one 950-gpm pump, one tank and the mentioned mains. Surge models were run with models containing 261, 143, 107, 16 and 3 pipes. The results indicate that each skeletonization tends to make the surge magnitudes greater and that the 16 and 3 pipe models, are significantly different than the original model. In the Summary the author's indicate that "overall transient head envelope does not drastically change until the number of pipes is reduced to less than 10 percent of the original system." For this work, the skeletonized surge model created for System #2 contains over 40 percent of the water distribution system, thus based on the Walski skeletonizing work it is assumed that little to no effect on the results will be due to skeletonizing.

2.6 Link Between Health Risks and Transients

In order to show the possibility of low and/or negative pressures within a water distribution system and how they can cause potential health effects, researchers have broken the problem into two areas: occurrences and effects of occurrence. In the paper titled "Occurrence of Transient Low and Negative Pressures in Potable Water Distribution Systems" (Gullick, et. al, 2004) multiple low and negative pressures are documented to exist within potable water distribution systems. The paper documents fifteen cases in which negative (i.e. below 0 psi) pressures were observed. Of these fifteen cases, the author of this work was personally involved in the site selection and recording of thirteen cases. Eleven of the thirteen cases were the result of pump shutdown operations and two cases were the starting and stopping of water cannons used in the cleaning of US military tanks. Of the eleven pump shutdown cases, ten were considered normal shutdowns, meaning that the operators at the treatment plant physically hit the "stop" button on the pump's motor control center starter. The single pump shutdown that was not intentional was caused by a lighting strike that caused several pumps to trip and shutdown at once.

In the paper titled "The Potential for Health Risks from Intrusion of Contaminants into the Distribution System from Pressure Transients (LeChevallier, et al, 2002) studies were done to determine if soil / water samples from areas around a water main could contain any type of contaminant that could be harmful to customers. The contaminants studied and identified were both chemical and biological in nature. Chemical contaminants, given enough accumulation can lead to acute toxicity while biological contaminants such as virus can cause infection with a single organism.

Given that occurrences of low and or negative pressure do exist and that soil sampling from soil surrounding water mains has indicated the presence of containments known to be harmful to humans, the link between transients and potential health effects is possible.

2.7 Conclusion to Literature Review

The papers cited and reviewed as part of this work indicate that low pressures and even negative pressures occur with some regularity in water distribution systems and that even something as common as a pump change operation can cause an occurrence. The citied papers also indicate that computer models do a good job of predicting transients within small water distribution models. Similarly it is well documented that soils and water samples taken from around water mains contain many harmful contaminants that could lead to adverse health effects for users of the water distribution system.

Due to the potential risk to human life, it is clear that the use of computer models that can predict areas affected by low and/or negative pressures will become an important tool to predict, mitigate and potentially prevent low and/or negative pressure occurrences.

Surge2000, surge-modeling software developed at the University of Kentucky is one such tool. This work will demonstrate how good a tool Surge2000 is in predicting low and/or negative pressures within a large complex water distribution system.

CHAPTER 3
RESEARCH OBJECTIVE AND APPROACH

3.1 Hypothesis

Current on the market computer transient models, such as Surge2000 developed by the Civil Engineering Software Center at the University of Kentucky, can be used to predict the magnitude, length (time) and locations of low and/or negative transients within large potable water distribution systems.

3.2 Objective

The research objective was to improve upon the surge modeling results of a previous study conducted through the American Water Works Research Foundation (AWWARF) Project No. 2686, which was conducted by Economic and Engineering Services, Inc., Tulane University, American Water Works Service Company and the University of Kentucky. Using the author's extensive knowledge of System #2 and access to boundary conditions, the objective was to truly determine if the surge model would correlate with the field data collected by the author for the AWWARF Project 2686. System #2 is a large complex water distribution system serving 350,000 people in the southeast United States. The system has 65 MGD of pumping capacity at two treatment plants, over 1500 miles of main and 12 storage tanks.

3.3 Research Approach

The approach for this study involves several distinct steps. Step one involves compiling the results from the AWWARF study and presenting them in tabular form for Modeling Scenarios 1 & 2 and for the Calibration Data (Scenario #4). Step two involves creating 24 hour extended period simulation hydraulic model runs using Pipe2000 for each of the three scenarios presented in the AWWRF study. The reason for performing the EPS runs is to confirm that the demands are correct in the model for the day and time modeled. The third step will be to produce four surge models, one for scenario 1, one for scenario 2, one for a lightning strike that occurred at WTP1 (Scenario #3), and a fourth and final model for the calibration model used in the AWWARF study (Scenario #4). These four surge models will be run to simulate the same events that occurred in real life and that were discussed in the AWWARF study.

The AWWARF study concluded “On average, steady state pressures calculated in the model were 20% lower than steady state pressures observed in the field. This was not consistent with the model calibration, for which steady-state pressures predicted by the model were often significantly larger than observed in the field.”(Friedman, 2004) By incorporating, more accurately the demand changes and system boundary conditions the models created for this study will have a better correlation with steady state conditions. This will allow for a true comparison of predicted transients pressure with field obtained values.

Presented on the following pages in Tables 3.1, 3.2 and 3.3 are the tabular results of the AWWARF Project 2686. Scenario #1 relates to the pump stopping operations performed at the WTP during pump drawdown tests. Scenario #2 relates to pump shutdown tests performed for the AWWARF study at one of the utilities pump storage facilities. Table 3.3 is the results of the calibration data based on drawdown tests at the WTP and was the data used to calibrate AWWARF’s surge model.

Table 3.1 – AWWARF Project 2686 Results of Scenario #1
Modeling scenario #1, field and model pressure results*

Pump # (Time of Closure)	Site #1			Site #3			Site #5		
	Pressure Range Max (psi)	Pressure Range Min (psi)	Pressure Drop (psi)	Pressure Range Max (psi)	Pressure Range Min (psi)	Pressure Drop (psi)	Pressure Range Max (psi)	Pressure Range Min (psi)	Pressure Drop (psi)
Pump 14 (55 sec)									
Field	**159**	**87**	**72**	**69**	**19**	**50**	**40**	**33**	**7**
Model (AWWARF Study)	148	86	62	51	12	39	30	23	7
Pump 10 (41 sec)									
Field	**158**	**95**	**63**	**67**	**27**	**40**	**39**	**32**	**7**
Model (AWWARF Study)	146	106	40	50	24	26	29	24	5
Pump 11 (25 sec)									
Field	**158**	**113**	**45**	**67**	**39**	**28**	**40**	**32**	**8**
Model (AWWARF Study)	144	52	92	49	-13	62	29	16	13

* Maximum pressures are equal to the initial steady state pressures for all sites.
Items in **bold** are field value

Table 3.2 – AWWARF Project 2686 Results of Scenario #2
Modeling scenario #2, field and model pressure results*

Pump # (Time of Closure)	Site #6			Site #5			Site #3		
	Pressure Range (psi)		Pressure Drop (psi)	Pressure Range (psi)		Pressure Drop (psi)	Pressure Range (psi)		Pressure Drop (psi)
	Max	Min		Max	Min		Max	Min	
Pump 1[1] (22 sec)									
Field	**98**	**37**	**61**	**45**	**33**	**12**	**74**	**68**	**6**
Model (AWWARF Study)	82	41	41	35	19	16	58	50	8
Pump 1[1] (24 sec)									
Field	**99**	**38**	**61**	**45**	**34**	**11**	**73**	**67**	**6**
Model (AWWARF Study)	83	42	41	35	20	15	58	50	8
Pump 1[1] (30 sec)									
Field	**100**	**40**	**60**	**47**	**35**	**12**	**77**	**70**	**7**
Model (AWWARF Study)	83	44	39	36	21	15	59	51	8
Pump 1[1] (52 sec)									
Field	**99**	**44**	**55**	**48**	**37**	**11**	**77**	**71**	**6**
Model (AWWARF Study)	84	50	34	36	23	13	59	53	6

* Maximum pressures are equal to the initial steady state pressures for all sites.
Items in **bold** are field values

[1] 300 Hp Pump

Table 3.3 – AWWARF Project 2686 Calibration Data

Start Time	Cause of Transient (Operating Condition)	Flow (Pre-condition) (MGD) Field	Flow (Pre-condition) (MGD) Model	Site#	Field Measurements: Pre-Condition Pressure (psi)	Field Measurements: Maximum Change in Pressure (psi)	Model Value: Pre-Condition Pressure (psi)	Model Value: Maximum Change in Pressure (psi)
7:51:34 AM	Shutdown of HS Pump 10	27.8	26.6	1	140	61	150	44
				2	63	52	72	42
				3	51	41	59	36
8:08:50 AM	Shutdown of HS Pump 14	19.3	19.6	1	132	78	135	79
				2	59	64	61	63
				3	48	51	50	52
9:04:52 AM	Shutdown of HS Pump 11	8.5	9.5	1	121	80	118	75
				2	51	60	48	61
				3	42	49	39	51
11:08:32AM	Startup of HS Pump 11	0	0	1	121	63	109	54
				2	52	57	40	47
				3	45	42	32	37
12:09:57 PM	Shutdown of HS Pump 11	7.6	5.5	1	135	113	119	98
				2	62	71	48	62
				3	53	58	40	54
12:47:44 PM	Startup of HS Pump 11	8.5	9.4	1	119	39	108	57
				2	50	38	39	54
				3	43	27	31	43
12:56:37 PM	Startup of HS Pump 14	18.9	19.6	1	125	39	119	55
				2	55	38	48	51
				3	47	26	40	39

Looking at Tables 3.1, 3.2 and 3.3 it is clear that the modeling results were somewhat non-successful in the fact that the calibration data correlated rather well but then scenarios #1 and #2 found in tables 3.1 and 3.2 did not correlate well at all. As an example, in Table 3.3, under the shutdown of pump 14 at 8:08 AM and pump 11 at 9:04 AM, the field measurements for both flow, pre-condition pressure and maximum pressure change are within 0.3 MGD and 3 psi or in terms of percentage about 2% for each. Two percent for all intensive purposes is considered to be excellent correlation for hydraulic modeling work. However, in Table 3.1, the surge models both over predicted and under predicted the pressures at sites 1, 3 & 5. In Table 3.2, the surge models under predicted at site 6 and over predicted at sites 3 and 5. In Table 3.3, the surge models both over and under predicted transient values as shown below:

- 7:51, under predicted,
- 9:04, both under and over predicted,
- 12:09, under predicted,
- 12:47, over predicted,
- 12:56, over predicted.

CHAPTER 4
METHODS

4.1 Field Work

The field work portion of this study involved three distinct activities. Activity one was conducted in the office and involved the review of water distribution maps, USGS Quad sheets, discussions with distribution system operators and pertinent operation data from the water utility. Activity two involved the setup and practice with the high-speed data recorders. Activity three involved the actual collecting of data during the transient scenarios.

4.1.1 Field Work Activity I

The purpose behind activity one was to determine the best location for monitoring for transient events to insure that time was not lost in tracking down locations. This task was accomplished in a couple of ways. First, a review of distribution maps was conducted to determine suitable locations. The areas targeted were locations downstream from any pumps (water treatment plants and booster pump stations), localized high points, areas in which low pressures complaints are commonly received and areas near large water users. This task yielded approximately 25 sites for consideration. Presented in Table 4.1 is a list of these 25 sites.

After the development of a list for potential sites the next step taken was to determine the potential for a surge event to occur at each of the sites. The main item looked at in this step was the magnitude of the change in velocity. The higher velocity the better the chance there would be to capture a transient event. Localized high points along long transmission mains were also investigated due to the potential for low and/or negative pressures to be present. Presented in Table 4.2 is a computation of the expected maximum velocity change at each site.

Table 4.1 – Initial Surge Site Locations

Site #	Location
1	Downstream High Service Pumps at 40 MGD Water Treatment Plant (WTP#1: 3-10 MGD & 3-8 MGD pumps)
2	Localized high point on north 30-inch transmission main downstream of Site #1
3	Localized high point on north 30-inch transmission main downstream of Site#2
4	Pump Storage Facility (YST: 1 MG tank & 2.5 MGD pump) that fills at high rate off 12-inch & 6-inch mains.
5	Highest point in distribution system. Location of existing SCADA monitored pressure vault. (STR) Routinely fields low pressure complaints from customer in area.
6	Pump Storage Facility (HRT: 3 MG tank, 2-3 MGD & 1-6MGD pumps)
7	Localized high point on south 30-inch transmission main downstream of Site#8
8	Localized high point on south 30-inch transmission main downstream of Site #1
9	Downstream High Service Pumps at 25 MGD Water Treatment Plant (WTP#2: 6 pumps from 4 to 12 MGD)
10	Localized high point on 8-inch in Eastern portion of system. Location of existing SCADA monitored pressure vault. (CHIL)
11	Localized high point on 8-inch in Eastern portion of system. Location of existing SCADA monitored pressure vault. (MAR)
12	Localized high point on 12-inch distribution main in SE portion of system.
13	Pump Storage Facility (CLY: 3 MG tank, 2-9 MGD pumps)
14	Pump Storage Facility (CX: 2-1 MG tanks, 1-2.5 MGD, 1-3 MGD pumps)
15	Pump Storage Facility (MER: 1 MG tank, 5 MGD pumps)
16	Pump Storage Facility (PM: 3 MG tank, 9 MGD pump)
17	Pump Storage Facility (HL: 0.2 MG tank, 2-0.5 MGD pumps)
18	Booster Pump Station (BH: 2-1.5 MGD pumps)
19	Booster Pump Station (NWT: 2-3 MGD & 1-2 pumps)
20	Booster Pump Station (LE: 2-0.4 MGD pumps)
21	Booster Pump Station (MTH: 2-0.4 MGD pumps)
22	Booster Pump Station (DEL: 0.9 MGD pump)
23	High demand customer (TOY) draws 5 MGD rate.
24	High demand customer (VER) draws 2 MGD rate.
25	High demand customer (UKY) draws 1.5 MGD rate.

Table 4.2 – Velocity Change per Surge Site

Site #	Max Flow (MGD)	Max Flow (cfs)	No. & size of pipes	Total Pipe Area (sf)	Max. Velocity Change (ft/s)
1	40.0	61.9	2 - 30” main	9.8	6.3
2	20.0	30.9	1 - 30” main	4.9	6.3
3	20.0	30.9	1 - 30” main	4.9	6.3
4	2.5	3.9	1 - 12”, 1 - 6” mains	1.0	3.9
5	Localized high point				
6	9.0	13.9	1 - 30” main	4.9	2.8
7	20.0	30.9	1 - 30” main	4.9	6.3
8	20.0	30.9	1 - 30” main	4.9	6.3
9	25.0	38.7	2-24”, 3-16”, 1 - 20”	12.7	3.1
10	Localized high point				
11	Localized high point				
12	Localized high point				
13	13.0	20.1	1 - 36” main	7.1	2.8
14	5.5	8.5	1 - 20” main	2.2	3.9
15	5.0	7.7	1 - 20” main	2.2	3.5
16	9.0	13.9	1 - 24” main	3.1	4.4
17	1.5	2.3	1 - 12” main	0.8	3.0
18	1.9	2.9	1 - 12” main	0.8	3.7
19	6.0	9.3	1 - 24” main	3.1	3.0
20	0.4	0.6	1 - 8” main	0.3	1.8
21	0.4	0.6	1 - 8” main	0.3	1.8
22	0.9	1.4	1 - 12” main	0.8	1.8
23	5.0	7.7	1 - 16” main	1.4	5.5
24	2.0	3.1	1 - 16” main	1.4	2.2
25	1.5	2.3	1 - 12” main	0.8	3.0

Table 4.2 can be narrowed down further based on a more in depth analysis of what may occur at each site. Based on a site-by-site analysis, the following text lists why sites were selected or not selected for potential transient location.

- Site 1: Selected because pumps are routinely started and stopped and power outages occasionally occur. Also the location for the transient model calibration and Scenario #1 in the AWWARF Study.

- Site 2: Selected because conditions at Site 1 may have impact at this site. This site is the first localized high point along the northern 30-inch transmission main that leaves the WTP at Site 1.
- Site 3: Selected because conditions at Site 1 may have impact at this site. This site is the second localized high point along the northern 30-inch transmission main that leaves the WTP at Site 1.
- Site 4: Selected because of potential impact that site may have on Site 5.
- Site 5: Selected because it is the highest point within the distribution system and the area where many low pressure complaints occur. This site is also monitored through SCADA and allows for verification of hydraulic (non transient) model performance.
- Site 6: Selected site because of potential impact that site may have on Site 5. Also the location for Scenario #2 in the AWWARF Study.
- Site 7: Selected because conditions at Site 1 may have impact at this site. This site is the first localized high point along the southern 30-inch transmission main that leaves the WTP at Site 1.
- Site 8: Selected because conditions at Site 1 may have impact at this site. This site is the second localized high point along the southern 30-inch transmission main that leaves the WTP at Site 1.
- Site 9: Not selected because velocity changes are rather low. Less than 3.1 feet per second (fps) at 25 MGD rate. Plant routinely runs at 12 MGD approximately 80% of time, thus velocity will usually be less that 1.5 fps.
- Site 10, 11 & 12: Not selected due to review of SCADA data that shows that these areas do not routinely witness transient events. Flows in these mains are generally low except in the case of a fire and without knowledge of when and where fire would occur it was decided not to use these three sites.
- Site 13: Not selected because the control valves used at this site have travel times greater than 2 minutes. Potential exists at this site, may be used for future studies.

- Site 14: Not selected because potential velocity change is not high. Normally velocity change will be around 1.5 fps. 2.8 fps value is when both pump storage facilities are operating at same time, which occurs only on maximum demand days.
- Site 15: Not selected, but potential exists at this site, may be used for future studies.
- Site 16: Not selected, but potential exists at this site, may be used for future studies.
- Site 17: Not selected because potential velocity change is not high. Normally velocity change will be around 1.5 fps. 3.0 fps value is when both pumps are operating at same time, which occurs only on maximum demand days.
- Site 18: Not selected because the control valves used at this site have travel times greater than 2 minutes and the pump motors are slowed at starting and stopping by variable frequency drives (VFD). Potential exists at this site if power outage occurs.
- Site 19: Not selected because potential velocity change is not high and because the control valves used at this site have travel times greater than 1 minute. Normally velocity change will be around 1.5 fps. 3.0 fps value is when two of the three pumps are operating at same time. This only occurs only on maximum demand days. Generator may be able to react fast enough during loss of power event to keep pumps running.
- Site 20: Not selected because this station is used less than 5% of the time.
- Site 21: Not selected because this station is used less than 5% of the time.
- Site 22: Not selected for this work, but pressure monitoring at this site with high-speed data loggers indicated transients occur at this location. The transients observed are high-pressure transients not low or negative pressure transients.

- Site 23: Not selected for this work but pressure monitoring at this site with high speed data loggers indicated transients occur at this location. The transients observed are high-pressure transients not low or negative pressure transients.
- Site 24: Not selected because demand use is sporadic.
- Site 25: Not selected because this large user is located within heart of distribution system with many looped feeds.

4.1.2 Field Work Activity II

The purpose behind activity two was to learn how to use the high-speed pressure recorders prior to their actual use in the field. The AWWARF Project 2686 team established protocols for data collection with input from this author. In the AWWARF Project 2686 Draft Final Report “Field Testing of Surge Model Prediction to Verify Occurrence of Distribution System Intrusion” (Economic and Engineering Services, et. al. 2002) and in the final report titled “Verification and Control of Pressure Transients and Intrusion in Distribution Systems”(Friedman, et. al. 2004) an entire section is devoted to the methodology used with the high-speed data recorders, data collection and transmission and need not be entirely repeated.

The high-speed pressure recorders were single-channel pressure transient data logger (Model RDL 1071L/3 Pressure Transient Logger; Radcom Technologies, Inc. Woburn, MA). In addition to the data loggers, software developed by Radcom was utilized to download and view data from the data loggers. The software utilized was Radlog for Windows V3.21.

The first test and setup of the data logger equipment occurred at the author’s home. The author installed the data logger on a house hose bids and had his son quickly open and close a bathroom faucet. The author then downloaded the data and viewed it with the Radlog software. Figure 4.1 is an image of the downloaded data.

Once he was satisfied that he knew how the equipment worked, he proceeded to perform a bench calibration test using a dead weight tester. A picture of the setup can be seen in Figure 4.2. This test helped to show a couple of important items. First, each Radcom unit has its own time clock, when the data is downloaded the unit's own time is used. If the three units are not fully synchronized than the speed at which the surge wave travels down a pipeline cannot de directly computed. Using the dead weight test we were able to determine the difference in time between the three units. The second item the dead weight test confirmed was the accuracy of the transmitter. In Figure 4.2 one can witness the layout of the three pressure transducers, note that the three transducers are at three different elevation, but all three units are within 6-inches or 0.2 psi. For this work that accuracy was sufficient.

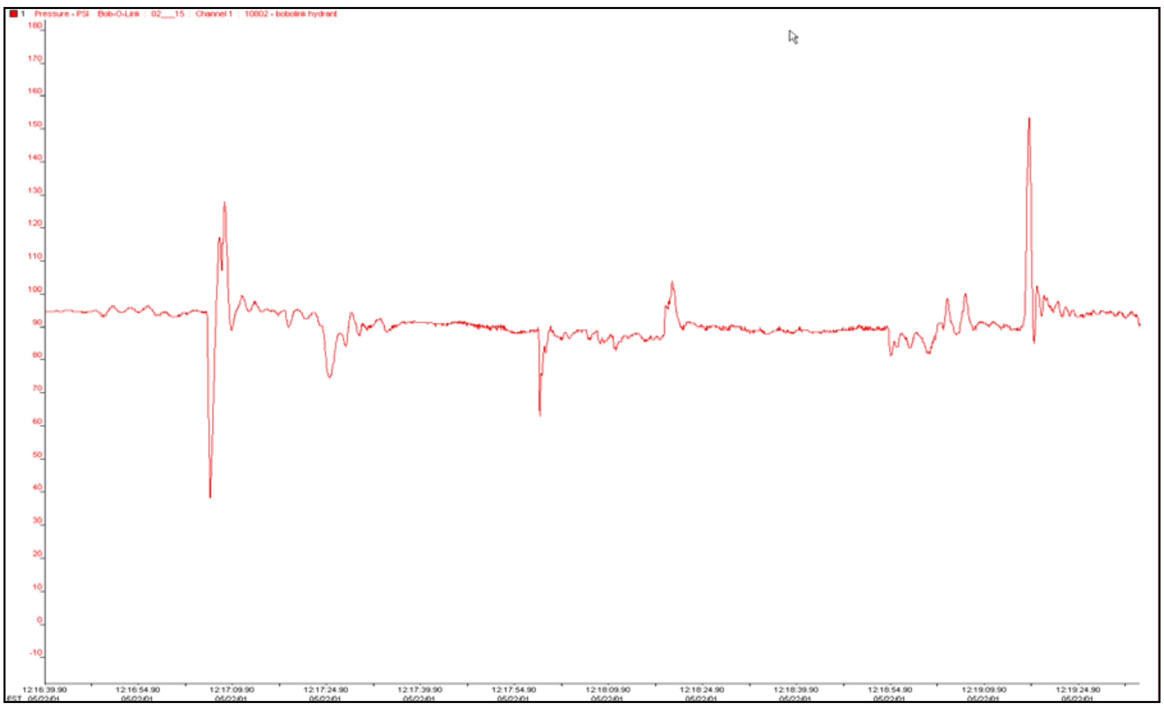


Figure 4.1 – Field Work Activity II Example of Test Data

Presented in Figure 4.3 is a graphical representation of the bench test. Notice that each of the units responds similarly and that the pressure displayed by each unit is the same after there is a chance for the signal to equalize. Also note the difference in time between the units. This difference in time is represented by the offset in the vertical portion of graphs.

Figure 4.2. – Field Work Activity II Bench Test

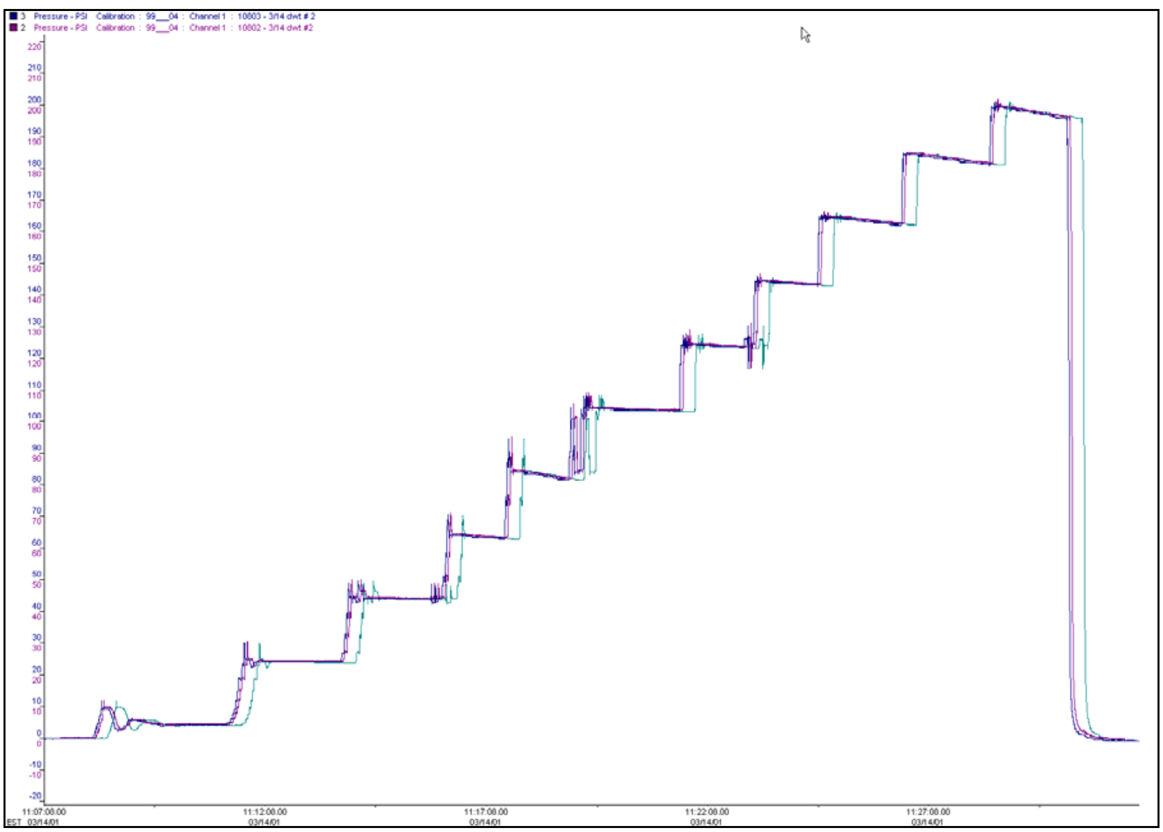


Figure 4.3 – Field Work Activity II Bench Test Results

4.1.3 Field Work Activity III

This field work activity involved the actual placement of the Radcom units in the field at the various selected sites. Figures 4.4 through 4.6 illustrate typical setups. The setup is simply comprised of two parts. The data logger (blue box in figures) and the pressure transmitter (silver device at end of wire). The pressure transmitter has a quick coupling that allows quick installation and removal. The second half of the quick coupling has a ¼” NPT male outlet. In Figure 4.4, this was connected to an existing 2” NPT male outlet corporation stop. A 2” x 1” reducer fitting and a 1” x ¼” bushing were used to finalize the connection. The transmitter was suspended from the ladder so entry into the confined space vault was not required every time data needed to be downloaded.

Figure 4.4 – Field Work Activity III Site 1 Location

Figures 4.5 and 4.6 show how the Radcom unit was mounted using an existing fire hydrant. In this case a special adapter piece was made to connect to the hydrant. The adapter was made by taking a 4-1/2” fire hydrant cap, making a 2” NPT tap and then using bushings to reduce down to the required ¼” NPT quick coupling. To allow air to be bled off when opening the hydrant, a tee and pet cock were installed. The hydrants were fully open during the monitoring to insure that the below ground weep holes were properly sealed.

Figure 4.5 – Field Work Activity III Site 2 Location

Figure 4.6 – Field Work Activity III Site 3 Location

4.2 Data Collection

Data collection is one of the most important items when constructing hydraulic models. Without good background data one is merely making an approximation as to what was going on within the distribution system at the time under investigation. Fortunately for this project, boundary condition data was available. The data was not necessarily easy to obtain; however, it was available. It is estimated to have taken nearly 120 hours to collect and compile the data for the different modeled scenarios. Presented below is a list of the data obtained, how it was manipulated, where it was used and how it is to be presented.

4.2.1 Daily Logs

Each water treatment plant (Site 1 and 9) keeps daily logs with a variety of information recorded manually by the water treatment plant operators. Pertinent information for this study included the starting and stopping time of each of the six high services pumps located at each plant, and each of the 13 pump storage and in-line booster pumps located throughout the distribution system. The daily logs are approximately 12” x 24” in size and are kept as permanent records at each of the treatment plants. Because there was no way to make an exact photocopy of each log, the pertinent information was recorded in a spreadsheet. Table 4.3 is an example of the recorded data for the high service pump for October 15, 2002. All other pump logs are attached in Appendix B.

Table 4.3 – Example of Pump Log Sheet used for Boundary Condition

HS Pump Log for Oct. 15, 2002			
Pump #	**Shift**		
	12am - 8am	**8am - 4pm**	**4pm - 12am**
WTP2-#6			
WTP2-#7	On	On	On
WTP2-#8			
WTP2-#9			
WTP2-#10			
WTP2-#11			
WTP1-#10		On 1:46, Off 1:59	
WTP1-#11		On 2:06, Off 2:26	
WTP1-#12			
WTP1-#13	On		On
WTP1-#14	On	Off 1:43, On 2:46	On
WTP1-#15	On		On

4.2.2 Chart Recorders

Each water treatment plant (site 1 and 9) uses circular chart recorders to continuously record high service flows and pressures. The chart recorders are used at each plant to measure 100% of the flow leaving each plant and one chart recorder per plant is used to measure discharge pressure. The circular chart recorders each receive electrical current signals from either a differential pressure transmitter (venturi meters) or a pressure transmitter (pressure). These transmitters are used to convert pressure into a 4 – 20 milli amp current signal. The chart recorder than graphically converts the 4 – 20 milli amp signal onto chart paper at the corresponding value. Figure 4.7, below is a picture of a circular chart recorder that is recording pressure. Copies of the individual chart records are not included in the Appendix because the values on the chart recorder were recorded at ½ hour intervals into a spreadsheet. An example of the recorded data is presented in Table 4.4.

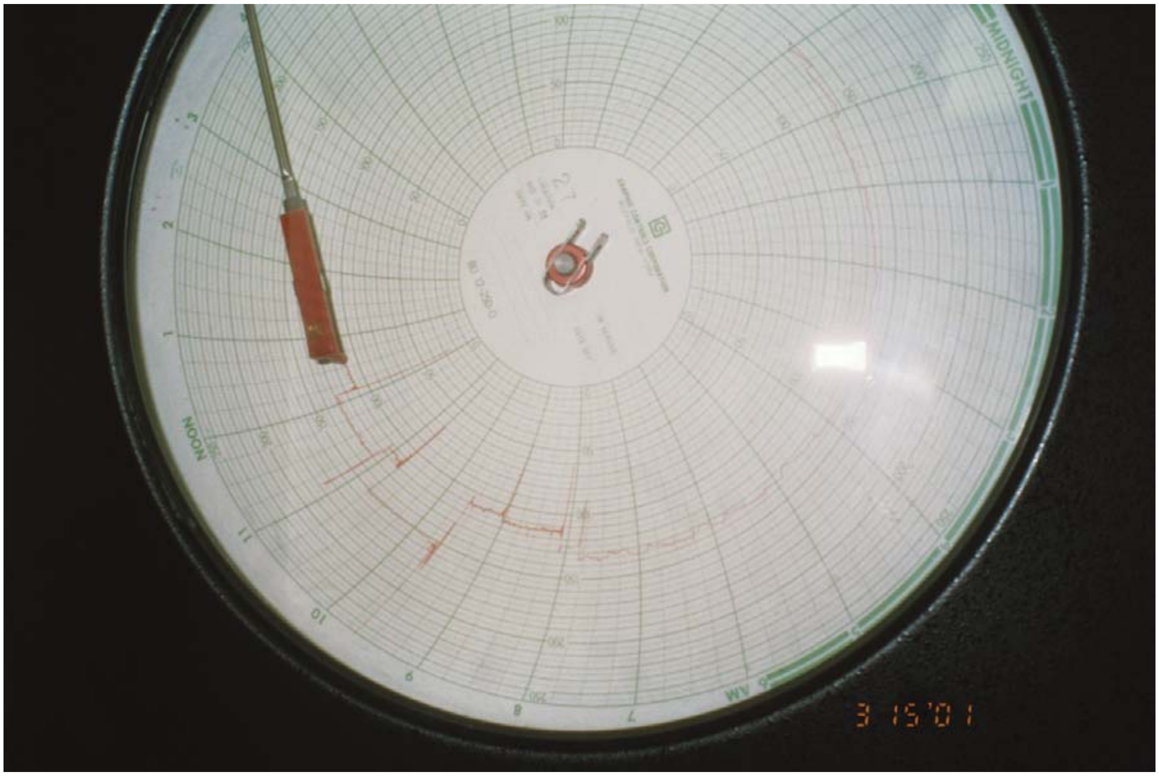


Figure 4.7 – Example of Circular Chart Recorder used for Boundary Condition

Table 4.4 – Example of Circular Chart Data used for Boundary Condition

Site 1 High Service Flows & Pressure					Site 9 High Service Flows & Pressure							
Date	hour	Meter 26 Flow (MGD)	Meter 37 Flow (MGD)	Pressure (psi)	Date	hour	Meter 1 Flow (MGD)	Meter 2 Flow (MGD)	SCADA Pressure (psi)	Site 9 Total (MGD)	Site 1 Total (MGD)	Total Production (MGD)
10/18/2002	0:00	14.4	16.6	142	10/18/2002	0:00	11.3	0.0	75	11.3	31.0	42.3
	0:30	14.4	16.5	142		0:30	11.6	0.0	74	11.6	30.9	42.5
	1:00	14.2	16.3	145		1:00	11.6	0.0	72	11.6	30.5	42.1
	1:30	14.1	16.3	144		1:30	11.4	0.0	72	11.4	30.4	41.8
	2:00	14.4	16.6	142		2:00	11.5	0.0	69	11.5	31.0	42.5
	2:30	14.3	16.6	142		2:30	11.5	0.0	70	11.5	30.9	42.4
	3:00	14.3	16.6	142		3:00	11.5	0.0	69	11.5	30.9	42.4
	3:30	14.6	15.9	140		3:30	11.5	0.0	68	11.5	30.5	42.0
	4:00	14.3	15.9	144		4:00	11.5	0.0	71	11.5	30.2	41.7
	4:30	13.8	15.4	153		4:30	10.6	0.0	79	10.6	29.2	39.8
	5:00	13.8	16.5	152		5:00	11.5	0.0	80	11.5	30.3	41.8
	5:30	14.3	16.3	152		5:30	11.5	0.0	70	11.5	30.6	42.1
	6:00	14.6	16.9	135		6:00	11.5	0.0	62	11.5	31.5	43.0
	6:30	14.6	16.3	140		6:30	11.5	0.0	66	11.5	30.9	42.4
	7:00	14.6	16.1	140		7:00	11.5	0.0	68	11.5	30.7	42.2
	7:30	14.4	15.9	145		7:30	11.4	0.0	72	11.4	30.3	41.7
	8:00	14.2	15.6	147		8:00	11.3	0.0	75	11.3	29.8	41.1
	8:30	14.3	15.7	146		8:30	11.5	0.0	72	11.5	30.0	41.5
	9:00	17.1	18.8	162		9:00	11.2	0.0	75	11.2	35.9	47.1
	9:30	17.0	18.9	162		9:30	11.2	0.0	76	11.2	35.9	47.1
	10:00	17.1	19.1	162		10:00	11.3	0.0	77	11.3	36.2	47.5
	10:30	17.6	19.3	158		10:30	11.3	0.0	74	11.3	36.9	48.3
	11:00	17.3	19.3	159		11:00	11.4	0.0	74	11.4	36.6	48.0
	11:30	17.3	19.3	158		11:30	11.3	0.0	74	11.3	36.6	48.0
	12:00	17.3	19.1	158		12:00	11.3	0.0	74	11.3	36.4	47.7
	12:30	17.3	19.1	158		12:30	11.2	0.0	75	11.2	36.4	47.7
	13:00	17.3	19.1	158		13:00	11.3	0.0	75	11.3	36.4	47.7
	13:30	17.3	19.1	158		13:30	11.1	0.0	76	11.1	36.4	47.6
	14:00	17.3	19.1	158		14:00	11.2	0.0	75	11.2	36.4	47.7
	14:30	17.3	19.1	160		14:30	11.2	0.0	74	11.2	36.4	47.7
	15:00	16.6	18.6	160		15:00	10.5	0.0	81	10.5	35.2	45.8
	15:30	16.6	18.6	166		15:30	11.1	0.0	82	11.1	35.2	46.3
	16:00	16.8	18.8	163		16:00	10.8	0.0	80	10.8	35.6	46.5
	16:30	16.8	18.8	163		16:30	10.8	0.0	79	10.8	35.6	46.5
	17:00	14.1	15.7	147		17:00	11.1	0.0	77	11.1	29.8	40.9
	17:30	14.2	15.7	147		17:30	11.1	0.0	76	11.1	29.9	41.0
	18:00	14.3	15.9	147		18:00	11.3	0.0	75	11.3	30.2	41.5
	18:30	14.4	16.0	147		18:30	11.3	0.0	73	11.3	30.4	41.7
	19:00	14.4	16.0	147		19:00	11.4	0.0	73	11.4	30.4	41.8
	19:30	14.4	16.0	147		19:30	11.2	0.0	75	11.2	30.4	41.6
	20:00	14.3	16.0	147		20:00	11.2	0.0	76	11.2	30.3	41.5
	20:30	14.3	16.0	147		20:30	11.1	0.0	76	11.1	30.3	41.4
	21:00	14.2	15.7	147		21:00	11.1	0.0	78	11.1	29.9	41.0
	21:30	14.2	15.7	147		21:30	11.1	0.0	77	11.1	29.9	41.0
	22:00	14.2	15.9	145		22:00	11.5	0.0	77	11.5	30.1	41.6
	22:30	14.2	15.9	144		22:30	11.4	0.0	75	11.4	30.1	41.5
	23:00	14.2	15.9	144		23:00	11.5	0.0	74	11.5	30.1	41.6
	23:30	14.2	15.9	144		23:30	11.5	0.0	73	11.5	30.1	41.6
		15.24	17.06				11.29			11.29	32.30	43.59

4.2.3 SCADA System Data

SCADA stands for Supervisory Control and Data Acquisition. SCADA is a very powerful tool used by many water treatment plant operators to monitor, control and record many activities within a water treatment and distribution system. Many older water distribution systems may not have a complete SCADA system and will have to rely on the chart recorders; however the system utilized by System #2 in the AWWARF Study had a fully implemented SCADA system.

Some of the items monitored by the SCADA system, which were used for this work, include tank levels at all twelve distribution storage tanks as well as pressures from twenty-five locations around the distribution system. Most of the pressure monitoring locations are used in conjunction with booster pumps. Section 4.5 will discuss the operation of the distribution system, but because System 2 in the AWWARF study is mostly a pressurized system with little floating storage, the pressure monitoring is extremely important.

The SCADA system is comprised of a master unit located at Site 9 (WTP2). This master unit polls each of the 27 remote terminal units located out in the distribution system. The data that is collected is recorded every 5 minutes to an ASCII file that is stored on the hard drive of each of the PC's that communicate with the master unit. During the time of this study the SCADA software used by the system operator was a late 1980's DOS version with no ability to link to more modern Windows based software (thus reason for storing in ASCII file). Using MS excel allowed the ASCII file to be converted into spreadsheet form that could in turn be graphed. Presented in Figures 4.8 and 4.9 are examples of the data which was converted to a spreadsheet and graph.

In Figure 4.8 and 4.9, the time of day is plotted on the horizontal axis starting at midnight with each vertical line representing one hour. In Figure 4.8 the vertical axis represents pressure in pounds per square inch (psi) with each horizontal line equal to 4 psi. In Figure 4.9 the vertical axis represents tank level in feet with each horizontal line equal to 1-foot. In Figures 4.8 and 4.9 notice the changes occurring between 2:30 PM and 3:30 PM. This is the time that Scenario #2 was conducted for the AWWARF project and the revised model runs used for this study. Note the change in pressures that occurred at Sites 5 & 6, an increase of approximately 8 – 12 psi. Note the tank level changes in Figure 4.9 during the same time interval. In Figure 4.9 an interesting item occurs. When the slope of the tank level is positive (increasing) the tank is filling. Likewise when the slope is negative the tank is drawing down via the booster pumps; however, notice the dip that occurs after every slope change. The reason for this is that the pressure transmitter that measures the tank level is not connected directly to the tank but rather to the pipe that fills or drains the tank. This has no bearing on this work but is mentioned to explain the dip in tank level. The dip is actually the velocity head component for the flow moving in the pipeline plus the sum of the headlosses required to push water into the tank or pull water from the tank. Notice that the fill rate is higher and thus the dip is bigger.

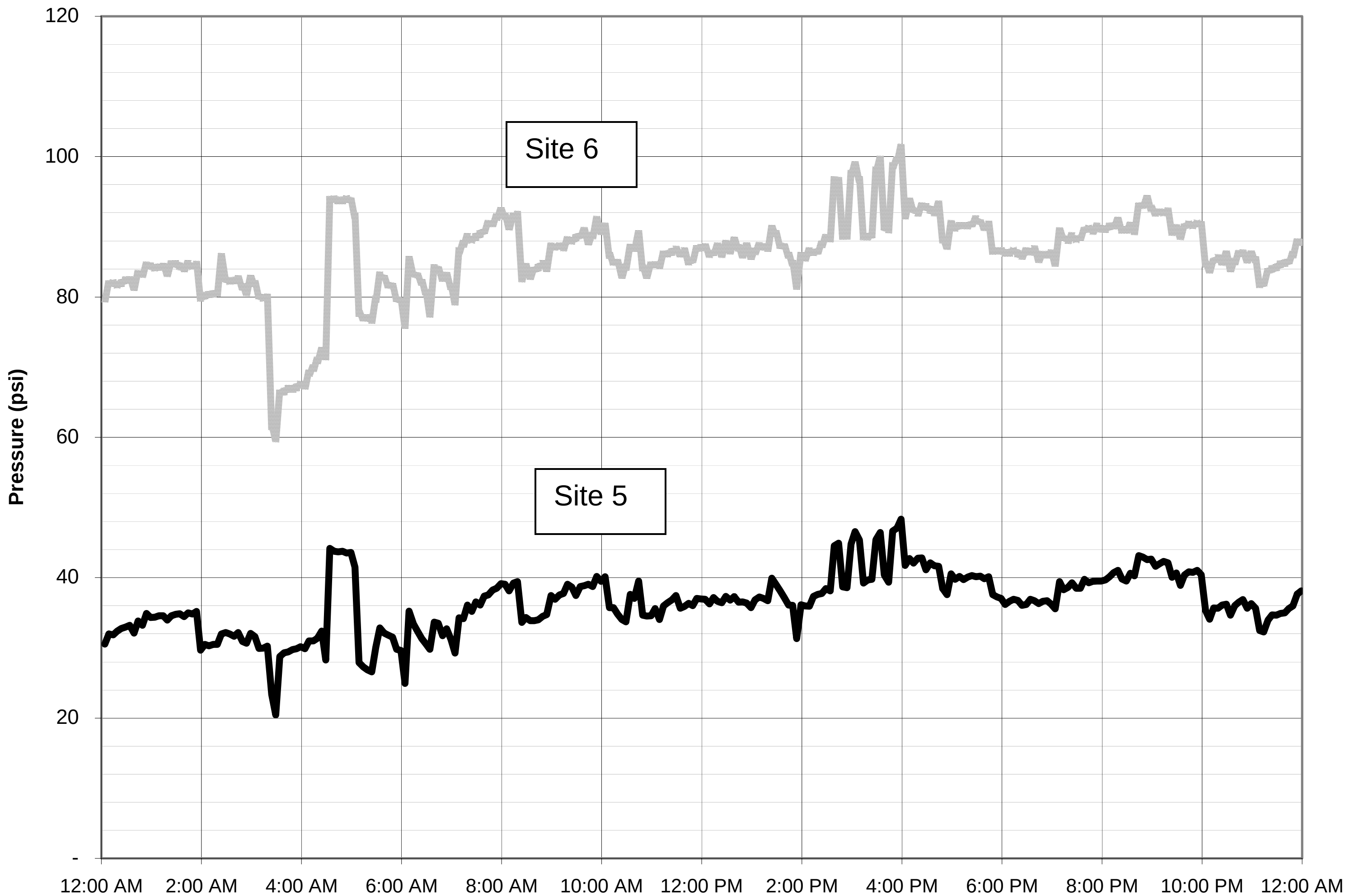


Figure 4.8 – Example of Converted Pressure Data used for Boundary Conditions

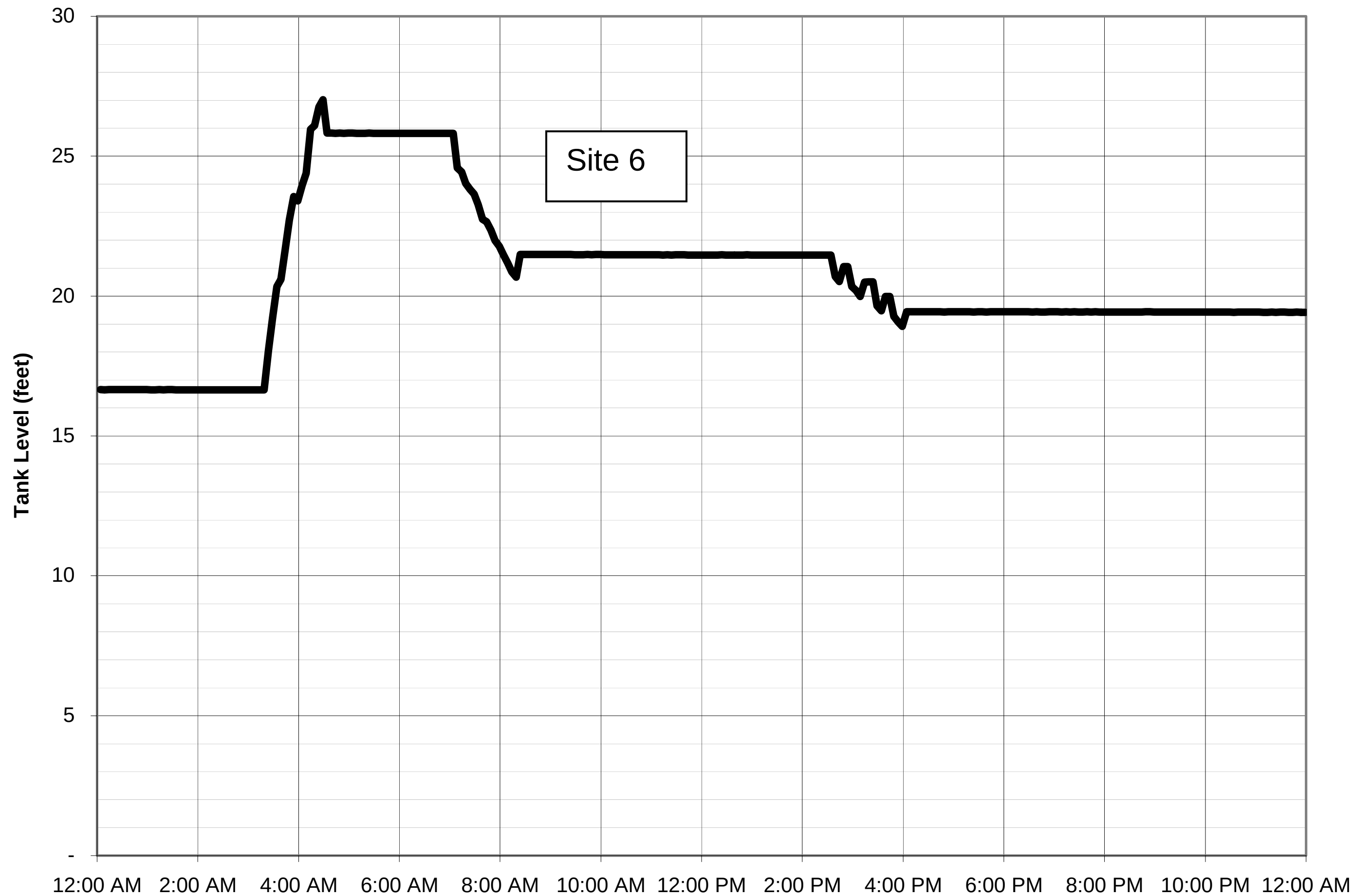


Figure 4.9 – Example of Tank Level Data used for Boundary Conditions

4.3 Computation of System Demands

After all the data is collected form the various sources the next step is to compile and organize the data. One of the most important factors in any hydraulic model is the allocation of demands. The base hydraulic model used for the AWWARF study and for this study was created by a third party consultant. The consultant created the model based on the maximum day of record for System #2, which occurred on June 13, 2000. In order to describe how the demands are allocated we must first understand how the consultant allocated the demands within the model. Presented below is a step-by-step procedure followed by the consultant. Please note that the work performed by the third party consultant was done under the direction and supervision of the System 2 engineering staff, which included the author.

- The consultant obtained the meter reading information for all customer accounts for the month of June 2000.
- The meter reading information is organized by area codes. Area codes are defined portions of the distribution system laid out on USGS Quad sheets and in the past loosely represented the area that the group of ten meter readers could read in one working day.
- The Information Services staff provided the total water used in a month per area code minus the 25 highest consumers. The unit of the water consumption was in 100 cubic feet increments and was converted to gallons per minute per area code.
- The Information Services staff also provided the total water used per month for the 25 highest consumers. The unit of the water consumption was in 100 cubic feet increments, which was converted to gallons per minute (gpm). See table 4.5 for breakdown of the 25 largest users. The largest consumer used 1097 gpm and the 25th largest consumer used 69 gpm. .

Table 4.5 – 25 Largest Water Users used for Boundary Conditions

Large User #	Customer Name	Usage (100 cf)	Usage GPD	Usage GPM
1	Uniform Service Co	4,430	110,467	77
2	Hospital 1	7,189	179,233	124
3	Municipal City 1	6,745	168,167	117
4	Municipal City 2	6,503	162,133	113
5	Fed. Gov. VA Hospital	5,662	141,167	98
6	Federal Medical Center	17,809	444,033	308
7	Manufacturer 1	11,421	284,767	198
8	Municipal City 3	6,402	159,633	111
9	Golf Course 1	4,049	100,967	70
10	Water District 1	24,754	617,200	429
11	Race Track Venue	6,821	170,067	118
12	Golf Course 2	4,040	100,733	70
13	Manufacturer 2	10,084	251,433	175
14	Manufacturer 3	9,623	239,933	167
15	Water District 2	3,953	98,567	68
16	Water District 3	8,393	209,267	145
17	Manufacturer 4	6,257	156,000	108
18	Hospital 2	7,005	174,667	121
19	Manufacturer 5	63,328	1,578,967	1097
20	Manufacturer 6	5,409	134,867	94
21	Public University	41,183	1,026,833	713
22	Public University	3,981	99,267	69
23	Public University	5,309	132,367	92
24	Public University	5,959	148,567	103
25	Public University	4,694	117,033	81
	Totals	281,003	7,006,333	4866

- The consultant created an ArcView shape file with the area code coverage lines and over laid this shape file over the hydraulic model. The hydraulic model is comprised of nodes (junctions) and pipes which connect the nodes. Demands are typically placed at each node to represent the consumption of water along a water main.

- The total demand per area code was divided by the total length of water mains within the area code.
- The demand at each node was then computed by taking the demand per foot of main and multiplying by ½ the total length of pipe that connected into each node. Thus nodes that had more pipe length connected to them received a proportionally higher demand, while nodes connected with shorter pipe lengths received a smaller demand.
- All the above demands were lumped together as demand type "R". The use of different demand types allows the use of different peaking factors.
- The 25 highest demand were added to the model at the exact location in which the use occurred and were given a different demand type.
- Average daily demands were not included in the model, as the month of June 2000 had demands greater than average.

The sum of the base demands allocated to the various nodes was 44.06 MGD. This value is 2 – 3 MGD higher than the normal daily average for System #2 but is reasonable for a summer month. The next and most important step is to determine the appropriate peaking factor for the base demands. A peaking factor is used to increase or decreased the amount of demand in the system throughout the day in order to more accurately simulate when the demand occurred. This approach makes sense because at night there would be less residential demand then during the day. The peaking factor is computed by taking the hourly change in tank level (computing flow from this change), adding that flow amount to the total production of the treatment plants and then dividing that amount by the 44.06 MGD base demand. For periods where tanks are filling the demand factor will typically be less than 1. Likewise when tanks are draining the peaking factor will typically be greater than 1. Presented in Figure 4.10 is an example of the peaking factors used in this study. A tabular list of peaking factors used is presented in Appendix B.

Figure 4.10 presents the peaking factors for four different days. June 13, 2000, was the base model prepared by the consultant. For all four data sets, in general the peaking factors are less than 1 early in the morning and in the late evenings. There exists a vast difference between the data for June 13, 2000 and the other data sets. The peak hour on June 13, 2000, occurred at 9:00 PM where as the other data sets did not have a pronounced peak hour. The July 4, 2001, data has a peak in the early morning that was due to the power failure that occurred to the Site 1 WTP. April 3, 2001, and October 18, 2002, had peaks that occurred in the late morning. Based on the shape of the demand factor curve, the importance of the proper demand factor becomes clear. Even within the same water system the shape of the demand curve can change dramatically over the course of a day and over the course of the year.

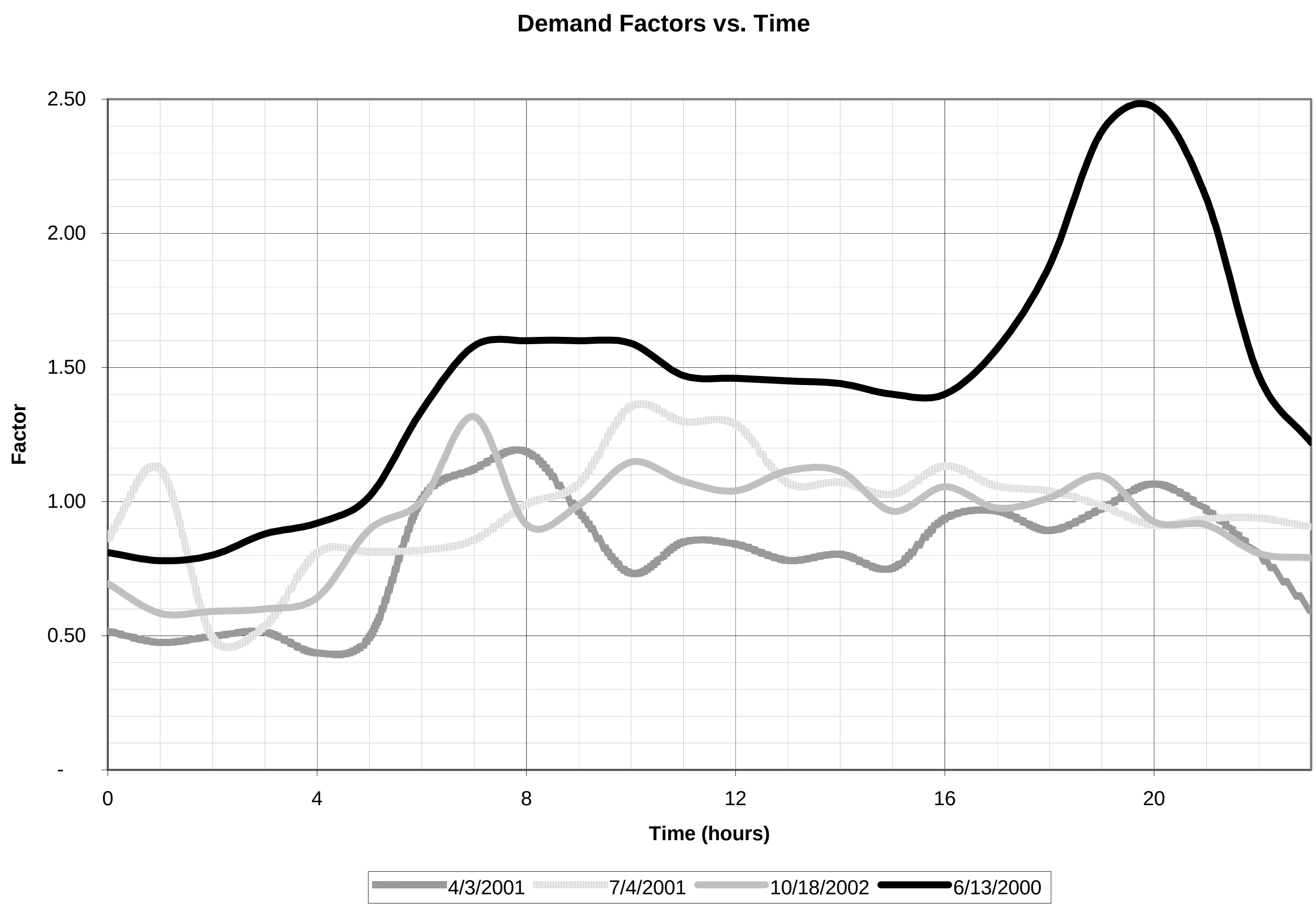


Figure 4.10 – Example of Peaking Factor Data used for Boundary Conditions

4.4 Computation of Celerity Values

After the allocation of demands the next most important requirement is the value of celerity (a.k.a. wave speed). Celerity is the speed of sound within the pipe and represents the speed the surge wave will travel in the main. Calculations for wave speed are presented in Appendix C. The general equation for wave speed is presented below

$$a = \sqrt{\frac{K/\rho}{1 + KDc_1/Ee}} : \qquad \text{Eq. (1)}$$

$$c_1 = 1 - \mu^2$$

where *K* is bulk modulus of fluid, ρ is density of fluid, *D* is diameter, μ is Poisson's ratio, *E* is Young's Modulus and *e* is wall thickness. Based on equation 1: for 30-inch reinforce concrete pipe (RCP) the wave speed or 'a' is equal to 3900 fps, for 30-inch ductile iron pipe (DIP) the wave speed equals 3700 fps, and for 6-inch polyvinyl chloride pipe (PVC) the wave speed is 1000 fps.

On page 64 of the Draft Final Report by Economic and Engineering Services, Inc., et. al. a value of 3000 fps was used for the wave speed. They indicated that this is a typical value for metal and reinforced pipes with small amounts of air. "The higher the wave speed used the greater the number of nodes with negative pressures occurred (System #5*)*" (Economic and Engineering Services, et. al., 2002). This makes sense, since a higher celerity ('a') value will produce a higher surge value.

In Dr. Wood's and Dr. Lingireddy's Analysis of System #2 Power Failure Event July 4, 2001", (Nov. 2001) several wave speeds were run and analyzed. Values analyzed were 3500, 2500 and 1500 fps. The amount of entrained air or trapped air greatly affects these values. In the final analysis Dr. Wood used a celerity value of 3500 fps for all pipes regardless of size or material.

As part of this work, different wave speeds were analyzed for each of the scenarios modeled. This was done to help determine what amount of entrained air gives the best results. The following runs were made.

- <u>All pipes with a wave speed value of 3000 fps</u>. This number is based on values used in the modeling of System #5 in the AWWARF study.

- All pipes with a wave speed value of 3800 fps. This value is the average value for all pipes based on material type and diameter in System #2 and was computed by computing taking the diameter times the length for each pipe size. Multiplying that value by the computed celerity for that size and type of pipe, adding up all individual pipe segments and then dividing that sum by the total inch-foot of all mains. The rounded value computed to be 3800 fps.
- All pipes will have a celerity value computed individually based on the size and type. This computation includes material, size and wall thickness. See Appendix C for values.
- Above values plus effect of entrained air. Take the value above (individual celerity values) and multiply by a factor to account for the effects of entrained air. The factor used is presented in Table 4.6.

Table 4.6 – Air Entrainment Factors used for Surge Model Runs

Material	Factor	% Air Entrainment
AC, DI, LJ, CI	0.51	0.10%
PVC, HDPE	0.86	0.10%
AC, DI, LJ, CI	0.35	0.25%
PVC, HDPE	0.75	0.25%
AC, DI, LJ, CI	0.26	0.50%
PVC, HDPE	0.61	0.50%
AC, DI, LJ, CI	0.20	1.00%
PVC, HDPE	0.50	1.00%

Presented below in Figure 4.11 is a graph showing the effects of entrained air on wave speed for Ductile iron pipe. Further computations are included in Appendix C.

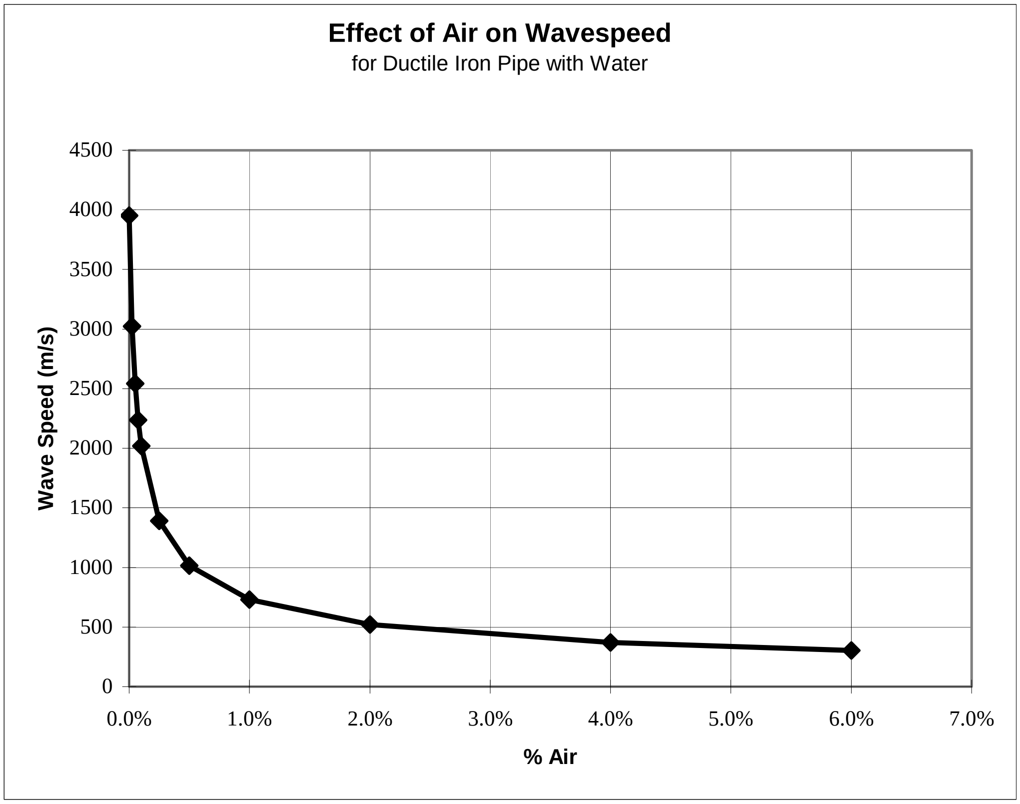


Figure 4.11 – Effect of Air Entrainment on Wave Speed

4.5 System #2 Description

System #2, provides domestic, commercial and industrial water service, as well as public and private fire protection, to all or portions of six counties in the southeast United States. The system also supplies wholesale water to three water districts, four municipalities and one army depot. The system's major source of supply is a river that in non-drought times yields greater than 72 MGD. System #2 also utilizes an 80 million gallon (MG) lake and a 600 MG reservoir. The reservoir is augmented by a raw water line from the before mentioned river.

The system has two treatment facilities: the WTP 1 (Site 1) has a reliable treatment capacity of 40 MGD and the WTP 2 (Site 9) has a reliable treatment capacity of 25 MGD. Peak hydraulic capacity of both plants is 80 MGD (50 MGD from WTP1 and 30 MGD from WTP2). Supply for WTP2 can be obtained from the river and from either the lake or reservoir. The lake is an emergency standby supply. Water from the reservoir is pumped three miles to the plant. WTP 2, which was the system's original treatment facility, consists of two open concrete settling basins, sixteen filters, a 600,000 gallon clearwell located under the filters, and a 450,000 gallon clearwell adjacent to the high service pumping station.

The raw water supply for the WTP 1 is the river. Six 12.5 MGD intake pumps, housed on an intake platform, pull water from the river and push it up a 400 foot cliff through a combination of three raw water lines (having diameters of 20-, 36- and 48-inches). Traveling screens on the intake structure prevent debris from entering the wet well and intake pumps. The water is delivered to the treatment plant and treated on site; a portion can be diverted to WTP 2 through a 30-inch ductile iron transmission main extending approximately 35,000 feet to the reservoir. Two 11.0 MGD transfer pumps are utilized to divert raw water to the reservoir.

WTP 1 consists of 10 hydrotreaters, each complete in themselves, affording mixing, flocculation, settling and filtration. Each is capable of filtering up to five million gallons per day. A clearwell under the pumping station holds approximately one million gallons of water and an adjacent above ground storage tank holds an additional two million gallons. Six high service pumps (ranging from 8 to 10 MGD) are used to pump finished water into the distribution system.

The distribution system consists of more than 1,500 miles of main ranging in size from 2-inch to 36-inch. These mains are of various materials, including: copper, gray cast-iron, ductile iron, asbestos cement (AC), polyvinyl chloride (PVC) and pre-stressed concrete. Mains 6-inches and larger in diameter comprise 89 percent of the system's total footage. The system has approximately 16.6 MG of water stored in tanks throughout the distribution system with an additional 4 MG under construction in 2004 and 2005. System #2 serves over 105,000 customers with an estimated population base of over 350,000. System #2 has over 6,500 public fire hydrants and over 1,600 backflow prevention devices.

Table 4.7 and Table 4.8 contain data pertaining to the make up of the distribution system piping.

Table 4.7 – System 2 Pipe Quantity by Size

Pipe Size (in)	Length (ft)	% Total
<2	283,556	3.54%
3	156,112	1.95%
4	397,098	4.96%
6	2,013,719	25.17%
8	3,258,595	40.73%
10	27,970	0.35%
12	1,061,641	13.27%
14	3,450	0.04%
16	267,749	3.35%
20	55,515	0.69%
24	318,039	3.98%
30	155,640	1.95%
36	624	0.01%
Totals	7,999,708	100%

Table 4.8 – System 2 Pipe Quantity by Material

Material Type	Length (ft)	% Total
AC	1,586,963	19.84%
CI	2,371,332	29.64%
CON	206,019	2.58%
DI	2,997,087	37.46%
GAL	16,808	0.21%
PEP	3,450	0.04%
PVC	817,424	10.22%
STL	625	0.01%
Total	7,999,708	100.00%

The acronyms used by System #2 for identification of pipe material are: AC - asbestos cement, CI - cast iron, CON – prestressed concrete pipe, DI – ductile iron, GAL - galvanized steel, PEP – high density polyethylene, PVC – polyvinyl chloride, STL – steel. From Tables 4.7 and 4.8 it would appear as though the 36-inch pipe should be steel, but that is not the case. The 624 feet of 36-inch pipe is all ductile iron, the 625-feet of steel segment is 30-inch in size.

4.6 Hydraulic Model Description

The hydraulic model for System #2 has evolved over the years from a simple 1000 pipe KYPipe model created in the early 1990's to a full blown 12,500 pipe Pipe2000 model finalized in 2001. The 12,500 pipe model included all mains 6-inch in diameter and larger and was developed to be used as pipe database and to allow the modeling of system performance when large transmission mains had to be shut down for relocation work.

The 12,500 pipe model has proven to be too cumbersome for routine planning activities due to the amount of time it takes to run, thus skeletonized versions of this model were created and calibrated. Two sizes were created: a 5,300 pipe version and a 2,500 pipe version. The model skelotonizing work was performed by Dr's. Wood and Linigireddy at the University of Kentucky under the direction of the author. The 2,500 pipe model was chosen to be used as part of the AWWARF project 2686 because it yielded excellent static and extended period simulation (EPS) runs in a reasonable amount of time. As a comparison, the 12,500 pipe EPS runs takes approximately 30 minutes to run whereas the 2,500 pipe EPS runs takes approximately 3 minutes to run using an Intel Pentium 4 mobile CPU running at 2 GHz.

As a comparison to the distribution system statistics presented in Table 4.7 and 4.8, the 2,500 pipe model is comprised of the lengths and sizes of pipe as presented in Table 4.9. The material comparison is in Table 4.10. Looking at Table 4.9, it is interesting to note that there is 1.21% of pipe smaller than 6-inches in diameter included in the model. A closer look at the model reveals that many of these lines are required for continuity purposes, serve areas of little or no demand and are not truly needed except for the fact that they provide model continuity. It is also interesting to note that the model includes more length of 14-inch diameter pipe than the system. The reason for this is that the field data is based on System #2 distribution map drawings and does not include some of the piping located within the property lines for the water treatment and booster pump station facilities. A closer look at the model reveals that a majority of the 14-inch piping is located on the discharge piping of the high service pumps at WTP1 and in yard piping at the pump storage facility Site 13.

Table 4.9 – System 2 Pipe Quantity Comparison Field versus Model

Pipe Size (in)	Field Length (ft)	Field % of Total	Model Length (ft)	Model % of Total
<2	283,556	3.54%	22,810	0.71%
3	156,112	1.95%	322	0.01%
4	397,098	4.96%	15,861	0.49%
6	2,013,719	25.17%	0	0.00%
8	3,258,595	40.73%	1,638,031	51.02%
10	27,970	0.35%	20,168	0.63%
12	1,061,641	13.27%	797,126	24.83%
14	3,450	0.04%	3,580	0.11%
16	267,749	3.35%	244,942	7.63%
20	55,515	0.69%	41,094	1.28%
24	318,039	3.98%	320,024	9.97%
30	155,640	1.95%	105,956	3.30%
36	624	0.01%	442	0.01%
Totals	7,999,708	100.00%	3,210,356	100.00%

Table 4.10 – System 2 Pipe Material Comparison Field versus Model

Material Type	Field Length (ft)	Field % Total	Model Length (ft)	Model % Total
AC	1,586,963	19.84%	585,650	18.24%
CI	2,371,332	29.64%	823,920	25.66%
CON	206,019	2.58%	195,959	6.10%
DI	2,997,087	37.46%	1,544,630	48.11%
GAL	16,808	0.21%	0	0.00%
PEP	3,450	0.04%	3,411	0.11%
PVC	817,424	10.22%	55,797	1.74%
STL	625	0.01%	989	0.03%
Total	7,999,708	100.00%		100.00%

In Table 4.9, it is also apparent that the model does not include as much of the small piping as the field does. Although one may think this to be wrong, if one examines the definition of transmission main and considers that a 12-inch in parallel with a 6-inch main has the equivalent diameter as a 12.7-inch diameter main then one will realize how insignificant the 6-inch and smaller mains become. The important part in reviewing the comparison of model to field data is the following.

- When comparing the 2,500 pipe model to the 12,500 pipe model one may assume that only 20% of the piping would be represented (2,500/12,500) in the smaller model; however, approximately 40% of the length of the entire distribution system is represented in the 2,500 pipe model.
- 8-inch piping makes up 40.7% of the distribution system and approximately 50% of this amount is represented in the 2,500 pipe model.
- 75% of the 12-inch pipe is represented in the 2,500 pipe model.
- 91% of the 16-inch pipe is represented in the 2,500 pipe model.
- 75% of the 20-inch pipe is represented in the 2,500 pipe model.
- 100% of the 24-inch pipe is represented in the 2,500 pipe model.
- 100% of the 30-inch pipe is represented in the 2,500 pipe model. (Field totals include approximately 50,000 feet of 30" raw water main that should not be included in the totals).

In Table 4.10 it is apparent that the model features more rigid pipe (AC, CI, DI & CONC) than the field. This is due to the fact that most of the PVC mains are not included in the model. PVC main is less rigid then AC, CI, DI & CONC, thus it is possible depending on the type of surge event that the model could produce higher surge magnitudes since the field will have more capacity to soften the transient event.

4.7 Pipe2000 EPS Runs

One of the recommendations of the AWWARF study was that more information was needed to verify system operations in terms of pump and tank status, better estimates of demands, etc. In order to verify that that the pump and tank status as well as the demands are correct for each of the modeling scenarios, EPS runs using Pipe2000 were conducted on the hydraulic model for each scenario day (April 4, 2001, October 15 & 18, 2002 and July 4, 2001). The main reason for performing these EPS runs was to verify that the changes

(changes includes pump and tank cycles as well as plant production rates) were correct throughout the day. Once the EPS model correlated well, then the hydraulic surge analysis would be started for the different scenarios. The main components needed to run the EPS models are: hourly demand factors, pipe changes, node changes, and pump changes. In Figure 4.10, a graph was presented illustrating the demand factors throughout the day. In Figure 4.9, a graph was presented of a typical pump storage tank level, graphs for all tanks are were used to determine start / stop times for tank filling and start and stop times for pump starts.

In Tables 4.3 and 4.4 examples are given of high service pump status as well as flows at each treatment plant. This information will be used as boundary conditions in the EPS model as well as verification of model performance. Presented in Figure 4.12 and 4.13 are comparisons of actual plant flow rates and pressures (from circular chart recorders) for WTP1 versus model predictions.

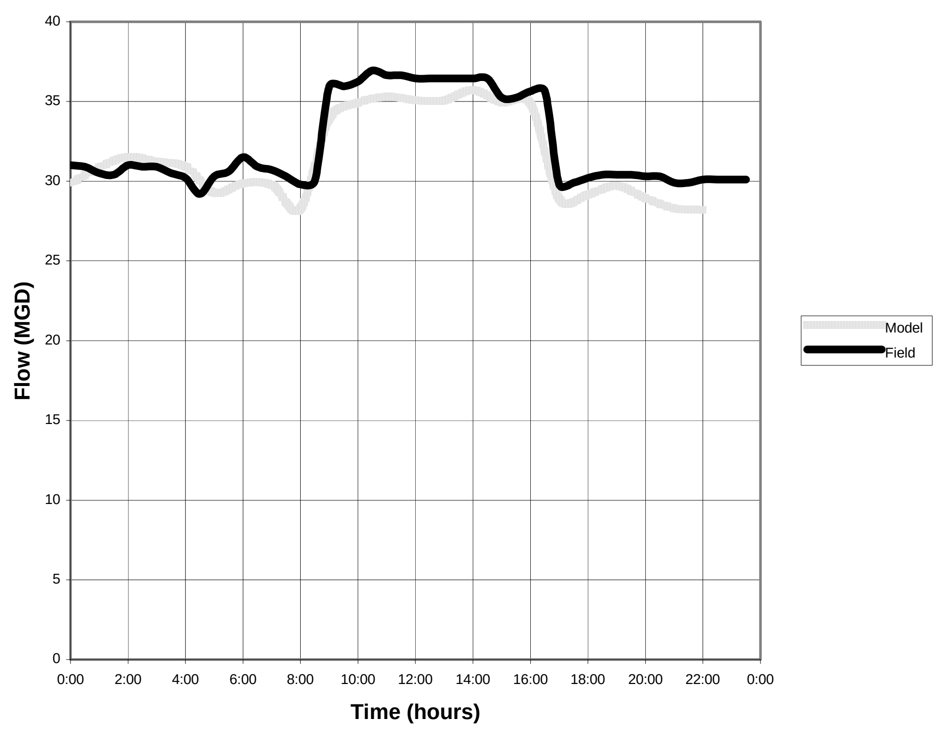


Figure 4.12 – EPS Runs Model vs. Field: Flows at WTP1 on 10-18-02

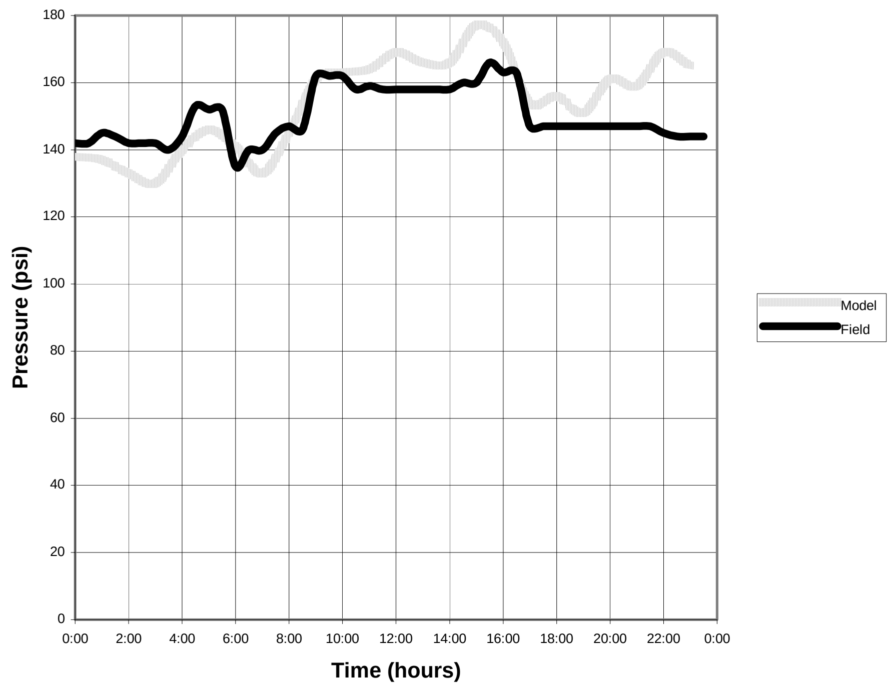


Figure 4.13 – EPS Runs Model vs. Field: Pressure at WTP1 on 10-18-02

4.8 Surge2000 Runs

Upon completion and verification that the hydraulic model simulated field conditions the next step was to create the surge model. The modifications needed to create the surge model from the EPS hydraulic model are quite simple. The following steps are taken to make the model change.

- First the appropriate demand factor must be used. Delete the demand factors for all cases in the EPS model except the value of the demand factor at the time of the actual test. Verify the values for wave speed have been assigned to each pipe within the model. As discussed in Section 4.4 different values for wave speed will be investigated.
- Verify that the changes of pump, pipe and tank status are correct for the time at which the surge model will be run. As an example, booster pumps are turned off and on through out the day in the EPS model. Since System #2 has little floating storage, the effect of missing a booster pump that is running can be great.

- Make the appropriate changes to the Pipe2000 setup files to insure that a Surge2000 model will be run. It is assumed that the reader of this paper knows how to operate Pipe2000 and Surge2000 and knows the steps needed in this part.
- Determine what event is causing the surge to occur and set that event up in Surge2000. This includes active valve settings and pump trips.
- In Surge2000 under *System Data >> Simulation Specs >>* set the demand calculation to pressure sensitive. During the surge model run this setting alters the demand at each node based on pressure. This approach is reasonable since higher pressures will increase demand at a given node.

In order to create the surge event, field data must be entered into the Surge2000 software. Because the pumps simulated all feature ball check valves as illustrated in Figure 4.16, two sets of input data is needed to create the surge event for each of the scenarios. Data set one is the closing speed of the ball check valve. The second data set is the point during the closing of the ball valve that power is shutoff to the pump's motor. Presented in Figure 4.14 and 4.15 are examples of valve closing and pump trip.

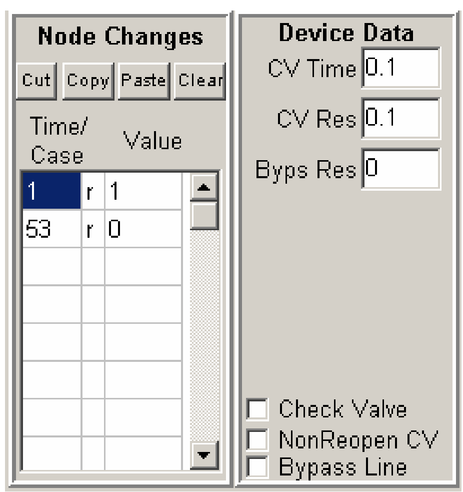


Figure 4.14 – Example of Active Valve Input Data used in Surge Model

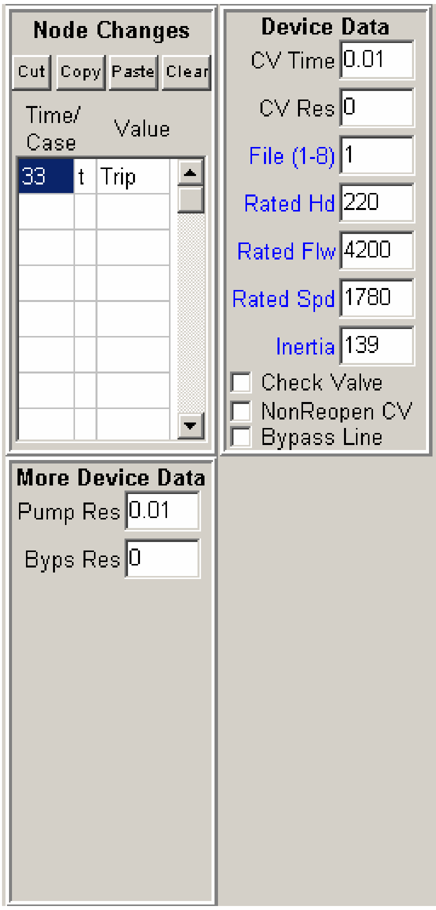


Figure 4.15 – Example of Pump Trip Input Data used in Surge Model

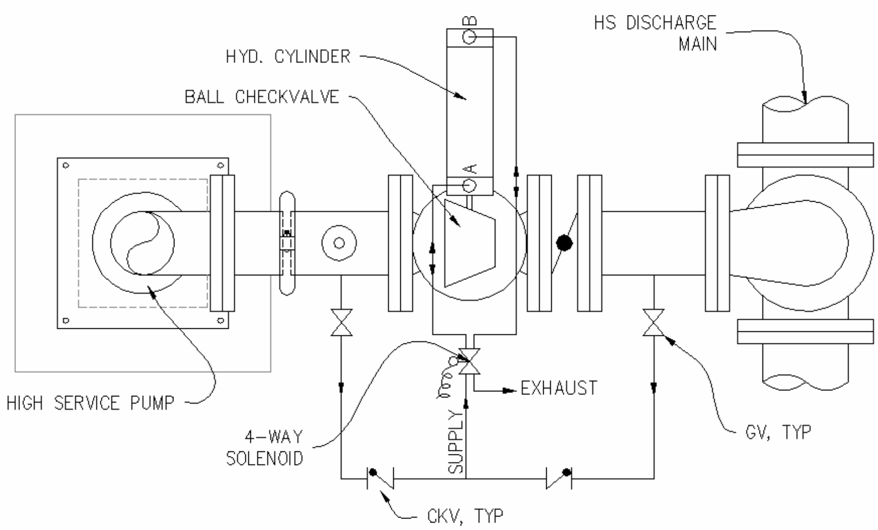


Figure 4.16 – Ball Check Valve Configuration

4.9 Surge2000 Pressure Contour Generation

One of the powerful features of Pipe2000 and Surge2000 is the ability to create pressure contours. The importance of this tool is that it provides a visual representation of pressures at a given time throughout the model. For surge modeling this feature can be used to determine the extent to which pressures drop below 20 psi or 0 psi. The 20 psi value is recommended and used as a lower bound for pressures in a distribution system due to the increase potential of a cross connection and is a minimum allowable pressure during flushing or fire flow events. Normally 30 psi must be available at all points in a distribution system during normal demands. See Section 5.9 for further discussion.

Presented in Figure 4.17 is an example of pressure contours around site 5 from a modeling scenario. In order to generate this minimum pressure contour map, in Surge2000, go to Map Settings > Emphasis/Contour > Node Contours.

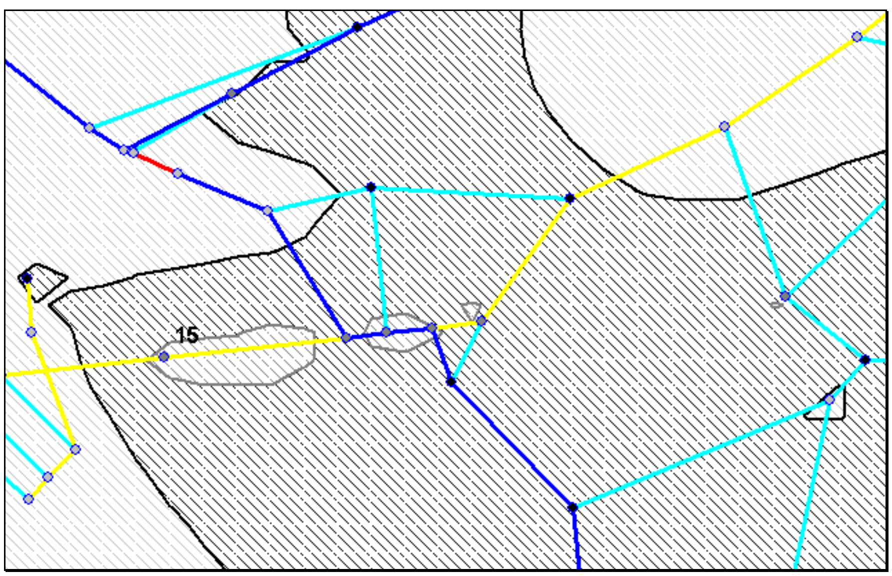


Figure 4.17 – Surge Model: Sample Results of Pressure Contours

The pressure contours in Figure 4.17 can then be overlaid with distributions maps or aerial photography to determine which areas should be targeted for surge mitigation equipment, backflow preventers or other devices. The tool allows engineers and utility owners to target areas for increased cross connection programs and increased water quality monitoring should a surge event be modeled that indicates high potential for a low and/or negative event.

4.10 Surge2000 Schematic

Presented in Figure 4.18 is a schematic of the model used for both the hydraulic extended period simulations (EPS) and the surge models. In order to give scale to the model, the direct distance from WTP1 (Site 1) to site 3 is approximately 22,200 feet. The direct distance between Site 3 and Site 6 is approximately 36,500 feet. Note that the distance via water mains is approximately 22,400 feet from Site 1 to Site 3 and 50,000 feet from Site 3 to Site 6 via the largest diameter mains.

In order to aid in the computation of the surge model, all intermediate nodes were removed, however, the actual lengths of the pipes remain the same. Also in order to create the skeletoninzed model, all long dead end lines were eliminated and the demand entered at the last non-deleted node. The effect this skeletonizing has on the model is not fully known, but given the fact that there were typically no changes in velocity occurring during these times, one could conclude that the skeletonizing of this model would have minor effects.

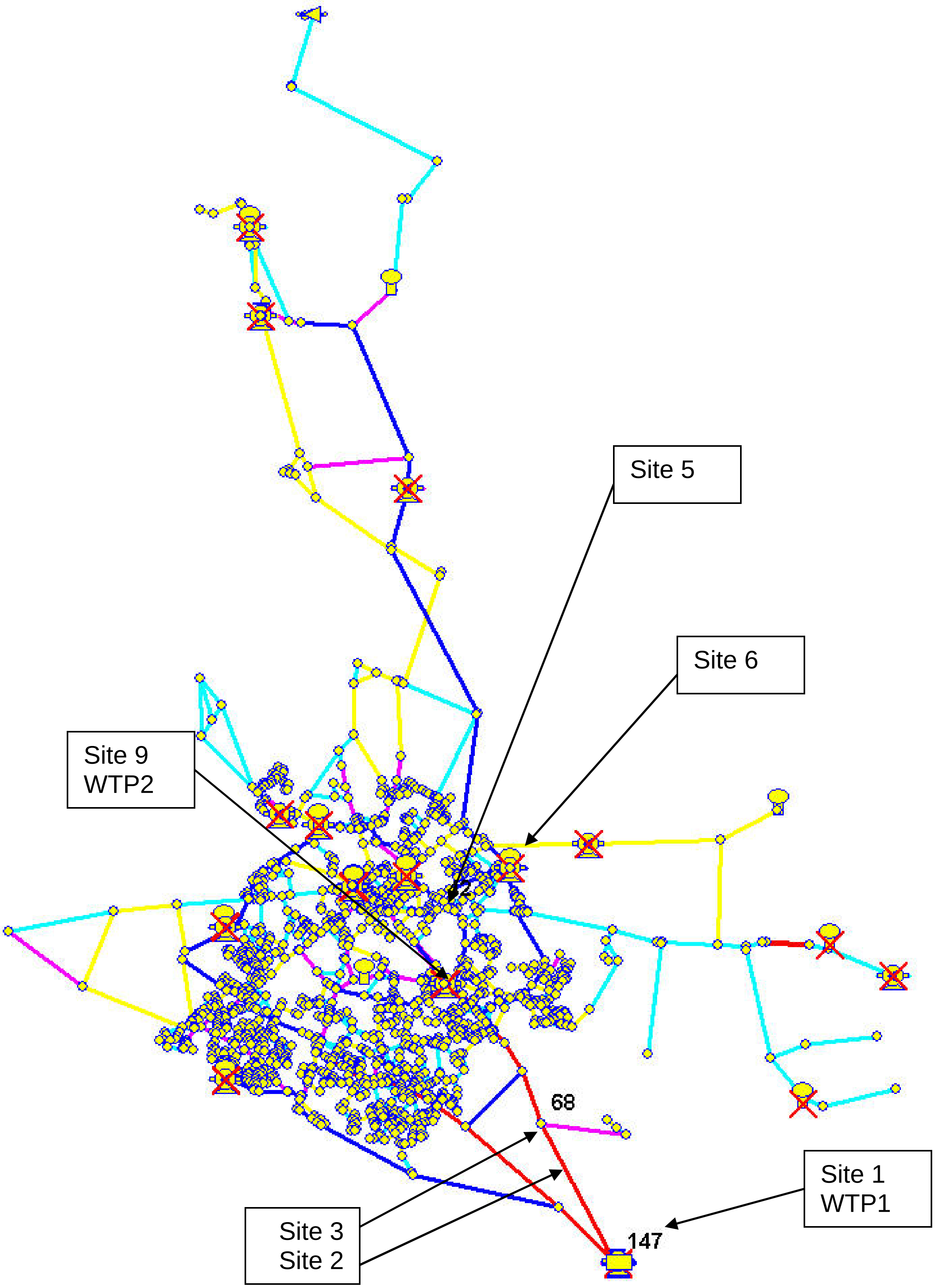


Figure 4.18 – Schematic of System 2 Surge Model

CHAPTER 5
RESULTS AND DISCUSSION

5.1 Introduction

In Section 3.2, the results of the AWWARF project 2686 were presented in Tables 3.1, 3.2 and 3.3. These tables showed how the modeling effort correlated with the author's collected field data. In the AWWARF study, the model was calibrated using the data available from the pump drawdown tests conducted at WTP1. Scenario #1 and Scenario #2 were then run using the calibrated models. In this work, each scenario was run as an independent model using actual boundary conditions verified by plant SCADA and WTP daily logs and then running multiple models with varying air entrainment and celerity values to determine the best data fit and draw conclusions from looking at the four independent runs. The boundary conditions used to set up each model was based on the information presented in Chapter 4, and consisted of tank levels, pump status, demand factors and the surge causing event. In each model, the pump files (Suter diagrams) were used as well as pressure sensitive demands.

Each scenario featured a different surge causing event. In scenario #1 a total of three surge events were modeled using six different values of celerity for each event. In total eighteen model runs were performed for scenario #1. In scenario #2 a total of four surge events were modeled using six different values of celerity for each event. In total twenty-four model runs were performed for scenario #2. In scenario #3 one surge event was modeled using six different values of celerity for each event. In total six model runs were performed for scenario #3. In scenario #4 three surge events were modeled using six different values of celerity for each event. In total eighteen model runs were performed for scenario #4.

As part of this work eleven surge models were created and run using six different values of celerity for each model. In total sixty-six models were created and run.

5.2 Scenario #1 Results

Scenario #1 involved the shutdown of three different high service pumps at WTP1 (Site #1) with different closing speeds on the pump check ball valves (see Figure 4.16). The surge wave was collected by field equipment located at Site #1, #3 and #5.

The first model created in this scenario involved the shutdown on a single 900 Hp - 10 MGD high service pump with a ball valve closing speed of 55 seconds and a pump trip (power cut off) occurring 26 seconds after the ball valve started to close. The second model created in this scenario involved the shutdown on a single 700 Hp - 8 MGD high service pump with a ball valve closing speed of 41 seconds and a pump trip (power cut off) occurring 21 seconds after the ball valve started to close. The third model created in this scenario involved the shutdown on a single 800 Hp - 8 MGD high service pump with a ball valve closing speed of 25 seconds and a pump trip (power cut off) occurring 16 seconds after the ball valve started to close.

Presented in Table 5.1 and 5.2 are the results of the runs for this scenario as well as the computed differences between the model and field data.

Table 5.1 – Tabular Surge Model Results for Scenario #1

Pump # (Time of Closure) Model Name	Site #1 Pressure Range (psi) Max	Site #1 Pressure Range (psi) Min	Site #1 Pressure Drop (psi)	Site #3 Pressure Range (psi) Max	Site #3 Pressure Range (psi) Min	Site #3 Pressure Drop (psi)	Site #5 Pressure Range (psi) Max	Site #5 Pressure Range (psi) Min	Site #5 Pressure Drop (psi)
Shutdown of Pump 14 (55 sec, 26 sec trip)									
Field Data	**159**	**87**	**72**	**69**	**19**	**50**	**40**	**33**	**7**
Model (AWWARF Study)	148	86	62	51	12	39	30	23	7
Model 1 - 3000fps all pipes	148	87	61	70	20	50	41	25	16
Model 2 - 3800 fps all pipes	403	-14	417	187	-14	201	42	-14	56
Model 3 - ws varies	148	80	68	71	16	55	41	22	19
Model 4 - 0.1% air all pipes	148	102	46	70	34	36	41	27	14
Model 5 - 0.25% air all pipes	149	97	52	70	32	38	40	31	9
Model 6 - 0.5% air all pipes	149	100	49	70	44	26	40	34	6
Shutdown of Pump 10 (41 sec, 21 sec trip)									
Field Data	**158**	**95**	**63**	**67**	**27**	**40**	**39**	**32**	**7**
Model (AWWARF Study)	146	106	40	50	24	26	29	24	5
Model 1 - 3000fps all pipes	149	91	58	69	24	45	41	27	14
Model 2 - 3800 fps all pipes	482	-14	496	208	-14	222	42	-14	56
Model 3 - ws varies	149	84	65	69	18	51	40	24	16
Model 4 - 0.1% air all pipes	149	104	45	69	36	33	41	29	12
Model 5 - 0.25% air all pipes	149	103	46	69	41	28	40	32	8
Model 6 - 0.5% air all pipes	149	107	42	69	46	23	40	34	6
Shutdown Pump 11 (25 sec, 16 sec trip)									
Field Data	**158**	**113**	**45**	**67**	**39**	**28**	**40**	**32**	**8**
Model (AWWARF Study)	144	52	92	49	-13	62	29	16	13
Model 1 - 3000fps all pipes	149	98	51	69	30	39	41	29	12
Model 2 - 3800 fps all pipes	470	-14	484	205	-14	219	42	-14	56
Model 3 - ws varies	149	93	56	69	25	44	40	27	13
Model 4 - 0.1% air all pipes	147	108	39	68	39	29	43	33	10
Model 5 - 0.25% air all pipes	148	74	74	68	25	43	43	34	9
Model 6 - 0.5% air all pipes	150	95	55	71	46	25	44	39	5

Table 5.2 – Difference between Model Results and Field Data for Scenario #1

Pump # (Time of Closure) Model Name	Site #1 Pressure Range Max (psi)	Site #1 Pressure Range Min (psi)	Site #1 Pressure Drop (psi)	Site #3 Pressure Range Max (psi)	Site #3 Pressure Range Min (psi)	Site #3 Pressure Drop (psi)	Site #5 Pressure Range Max (psi)	Site #5 Pressure Range Min (psi)	Site #5 Pressure Drop (psi)
Shutdown of Pump 14 (55 sec, 26 sec trip)									
Model (AWWARF Study)	-11	-1	-10	-18	-7	-11	-10	-10	0
Model 1 - 3000fps all pipes	-11	0	-11	1	1	0	1	-8	9
Model 2 - 3800 fps all pipes	244	-101	345	118	-33	151	2	-47	49
Model 3 - ws varies	-11	-7	-4	2	-3	5	1	-11	12
Model 4 - 0.1% air all pipes	-11	15	-26	1	15	-14	1	-6	7
Model 5 - 0.25% air all pipes	-10	10	-20	1	13	-12	0	-2	2
Model 6 - 0.5% air all pipes	-10	13	-23	1	25	-24	0	1	-1
Shutdown of Pump 10 (41 sec, 21 sec trip)									
Model (AWWARF Study)	-12	11	-23	-17	-3	-14	-10	-8	-2
Model 1 - 3000fps all pipes	-9	-4	-5	2	-3	5	2	-5	7
Model 2 - 3800 fps all pipes	324	-109	433	141	-41	182	3	-46	49
Model 3 - ws varies	-9	-11	2	2	-9	11	1	-8	9
Model 4 - 0.1% air all pipes	-9	9	-18	2	9	-7	2	-3	5
Model 5 - 0.25% air all pipes	-9	8	-17	2	14	-12	1	0	1
Model 6 - 0.5% air all pipes	-9	12	-21	2	19	-17	1	2	-1
Shutdown Pump 11 (25 sec, 16 sec trip)									
Model (AWWARF Study)	-14	-61	47	-18	-52	34	-11	-16	5
Model 1 - 3000fps all pipes	-9	-15	6	2	-9	11	1	-3	4
Model 2 - 3800 fps all pipes	312	-127	439	138	-53	191	2	-46	48
Model 3 - ws varies	-9	-20	11	2	-14	16	0	-5	5
Model 4 - 0.1% air all pipes	-11	-5	-6	1	0	1	3	1	2
Model 5 - 0.25% air all pipes	-10	-39	29	1	-14	15	3	2	1
Model 6 - 0.5% air all pipes	-8	-18	10	4	7	-3	4	7	-3

From Tables 5.1 and 5.2 it is seen that using a wave speed of 3800 ft per second for all pipes yields poor results. In presenting the results, the AWWARF study, listed the maximum pressure observed, the minimum pressure observed as well as the difference between these two values. In terms of predicting negative pressure, which could potentially lead to intrusion, the most important results are the minimum pressure computed by the model. The magnitude sheds some light into how well the model tracts with the field data, but the most important result is the minimum pressure observed and the length of time for the low and or negative pressure.

In review of Table 5.1 one can see that the AWWARF study predicted lower pressures than the field data for each of the scenarios except at Site #1 on shutdown of Pump10. In comparison, each of the model runs as a part of this work, yielded results both above and below the field data. This shows that the value chosen for celerity is critical in the accurate modeling of surge events.

Several methods were explored to determine which model fit the field test data the best. In the end, because each site or data point is equally important, a simple average of the absolute value of the percentage difference was computed and compared. The formula below was used to determine best-fit percentage:

$$\text{Best Fit} = 1 - \left[\frac{\sum_{1}^{\text{n,\# of date sets}} \left(abs\left[1 - \left(\frac{MR_n}{FR_n} \right) \right] \right)}{n} \right] \qquad \text{Eq. 2}$$

where *MR* is model results, *FR* is field results, *n* is the number of data set. The absolute value was used to insure that data sets that had 50% correlation on one test and 150% correlation on another test did not average out to be 100%. In addition, a comparison was made between the magnitude of pressure drop recorded in the field versus the magnitude of the pressure drop predicted in the model. To compute this value the sum of the pressure drop magnitudes were added up for each model and divided by the sum of the pressure drop magnitudes recorded in the field.

Presented in Table 5.3 is a comparison of percent fit for each model run using the above mentioned average fit approach and magnitude of pressure drop approach. Looking at the two computed correlation values, we find that for the 55 second shutdown model 1 and 3 yielded better results than the AWWARF study. In the 41 second and 25 second shutdowns, models 1, 3, 4, 5 & 6 all yielded better results than the AWWARF study. Of all the models in each of the shutdowns model 1 fit the best overall. This equates to a model with a celerity or wave speed value of 3000 fps that would be between 0% air entrainment and 0.1% air entrainment.

Also presented in Table 5.3 is the length of time in which pressures were below 20 psi for the best-fit models. As in the field the model did not predict any amount of time where the pressure at any of the sites was less than 20 psi.

Table 5.3 – Best-Fit Computation for Scenario #1 Surge Results

Pump # (Time of Closure) Model Name	Max - Min Best Fit	Pressure Drop Best Fit	Time less than 20 psi(*) (sec)
Shutdown of Pump 14 (55 sec, 26 sec trip)			
Model (AWWARF Study)	79%	84%	
Model 1 - 3000fps all pipes	**93%**	**98%**	**0**
Model 2 - 3800 fps all pipes	-27%	522%	
Model 3 - ws varies	88%	110%	
Model 4 - 0.1% air all pipes	79%	74%	
Model 5 - 0.25% air all pipes	84%	77%	
Model 6 - 0.5% air all pipes	74%	63%	
Shutdown of Pump 10 (41 sec, 21 sec trip)			
Model (AWWARF Study)	82%	65%	
Model 1 - 3000fps all pipes	**93%**	**106%**	**0**
Model 2 - 3800 fps all pipes	-39%	704%	
Model 3 - ws varies	86%	120%	
Model 4 - 0.1% air all pipes	89%	82%	
Model 5 - 0.25% air all pipes	88%	75%	
Model 6 - 0.5% air all pipes	83%	65%	
Shutdown Pump 11 (25 sec, 16 sec trip)			
Model (AWWARF Study)	50%	206%	
Model 1 - 3000fps all pipes	91%	126%	
Model 2 - 3800 fps all pipes	-33%	937%	
Model 3 - ws varies	87%	140%	
Model 4 - 0.1% air all pipes	**96%**	**96%**	**0**
Model 5 - 0.25% air all pipes	85%	156%	
Model 6 - 0.5% air all pipes	87%	105%	

(*) at Site 5, actual field time below 20 psi was 0 seconds for each case.
value in **bold** is best-fit

Presented in Figures 5.1, 5.2 and 5.3 are graphs that illustrate how the best-fit data for each pump shutdown correlated with the field data. Each figure shows the field and model pressure at sites 1, 3 and 5.

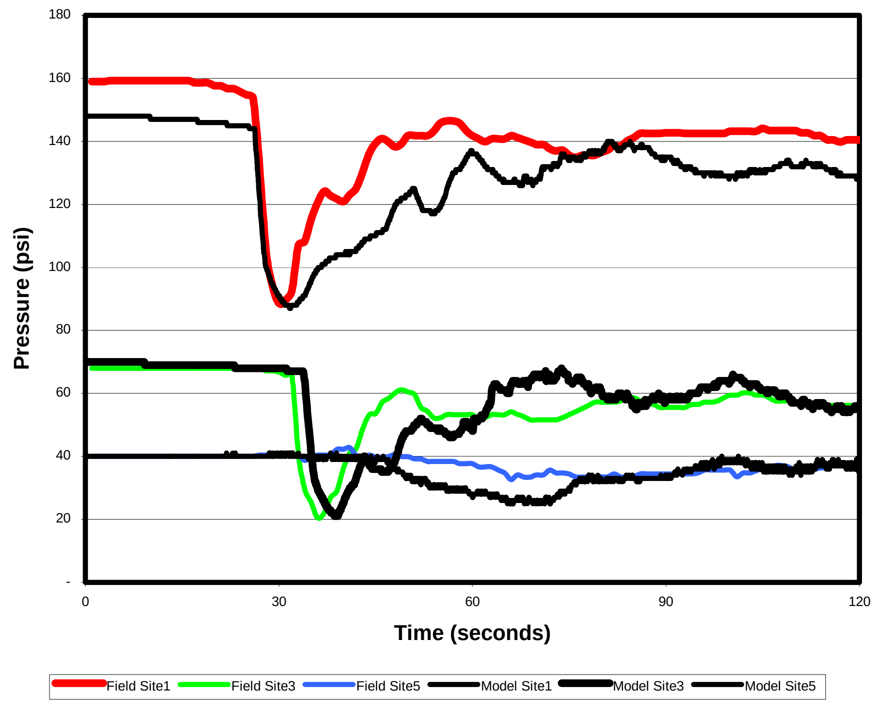


Figure 5.1 – Scenario #1 Pump 14 Shutdown Field vs. Model 1

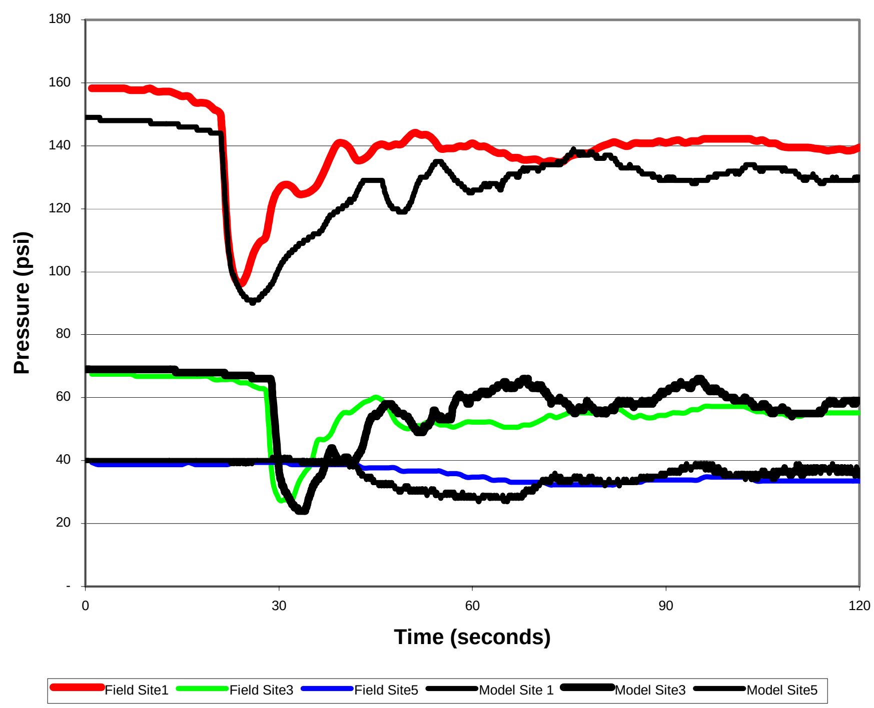


Figure 5.2 – Scenario #1 Pump 10 Shutdown Field vs. Model 1

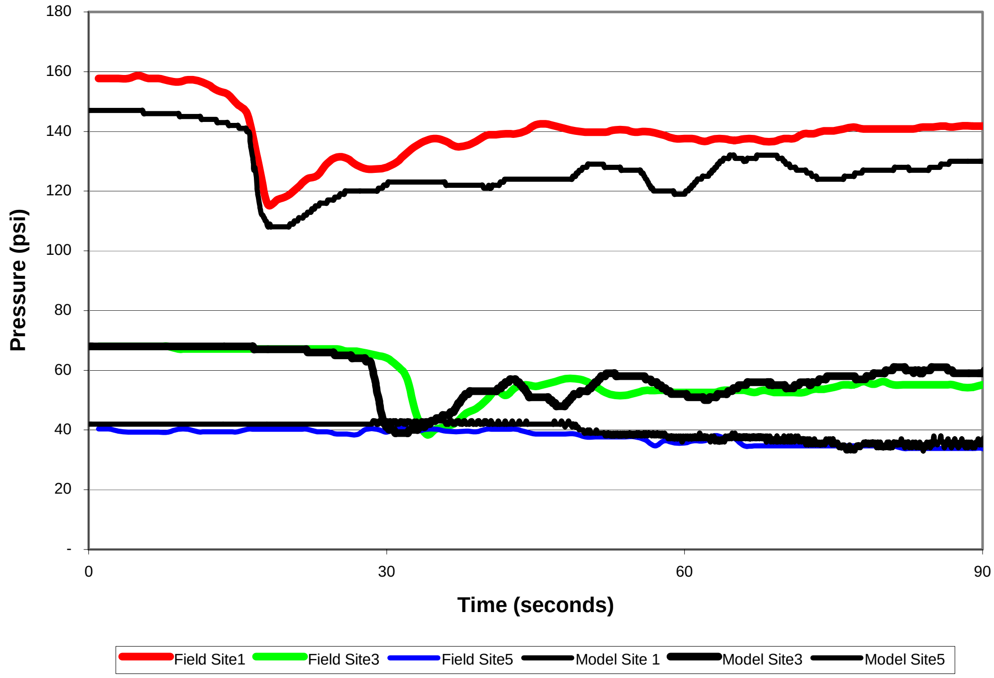


Figure 5.3 – Scenario #1 Pump 11 Shutdown Field vs. Model 4

5.3 Scenario #2 Results

Scenario #2 involved four pump shutdowns of one three hundred horsepower 6 MGD booster pump at a pump storage facility located out in the distribution system (Site #6). Each shutdown was with a different ball valve closing speed. The surge wave was collected by field equipment located at Site #6, #5 and #3.

The first model created in this scenario involved the shutdown of a single - 6 MGD booster pump with a ball valve closing speed of 22 seconds and a pump trip (power cut off) occurring 15 seconds after the ball valve started to close. The second model created in this scenario involved the shutdown of the same booster pump with a ball valve closing speed of 24 seconds and a pump trip (power cut off) occurring 15 seconds after the ball valve started to close. The third model created in this scenario involved the shutdown of the same booster pump with a ball valve closing speed of 31 seconds and a pump trip (power cut off) occurring 19 seconds after the ball valve started to close. The fourth model created in this scenario involved the shutdown of the same booster pump with a ball valve closing speed of 52 seconds and a pump trip (power cut off) occurring 33 seconds after the ball valve started to close.

Presented in Table 5.4 and 5.5 are the results of the runs for this scenario as well as the computed differences between the model and field data

From Tables 5.4 and 5.5 it is seen that using a wave speed of 3000 fps and 3800 fps for all pipes yields poor results. In presenting the results, the AWWARF study, listed the maximum pressure observed, the minimum pressure observed as well as the difference between these values. In terms of predicting negative pressure that could potentially lead to intrusion, the most important results are the minimum pressure computed by the model. The magnitude sheds some light into how well the model tracts with the field data, but the most important result is the minimum pressure observed and the length of time for the low and or negative pressure.

Table 5.4 – Tabular Surge Results for Scenario #2

Pump # (Time of Closure) Model Name	Site #6 Pressure Range Max (psi)	Site #6 Pressure Range Min (psi)	Site #6 Pressure Drop (psi)	Site #5 Pressure Range Max (psi)	Site #5 Pressure Range Min (psi)	Site #5 Pressure Drop (psi)	Site #3 Pressure Range Max (psi)	Site #3 Pressure Range Min (psi)	Site #3 Pressure Drop (psi)
Pump 1 (22 sec, 16 sec trip)									
Field	**98**	**37**	**61**	**45**	**33**	**12**	**74**	**68**	**6**
Model (AWWARF Study)	82	41	41	35	19	16	58	50	8
Model 1 - 3000fps all pipes	153	-14	167	58	-10	68	75	60	15
Model 2 - 3800 fps all pipes	118	-14	132	59	-7	66	75	59	16
Model 3 - ws varies	97	29	68	44	10	34	75	63	12
Model 4 - 0.1% air all pipes	97	38	59	45	20	25	74	67	7
Model 5 - 0.25% air all pipes	99	32	67	39	21	18	74	69	5
Model 6 - 0.5% air all pipes	99	43	56	47	26	21	74	70	4
Pump 1 (24 sec, 16 sec trip)									
Field	**99**	**38**	**61**	**45**	**34**	**11**	**73**	**67**	**6**
Model (AWWARF Study)	83	42	41	35	20	15	58	50	8
Model 1 - 3000fps all pipes	163	-14	177	58	-10	68	75	60	15
Model 2 - 3800 fps all pipes	114	-14	128	54	-14	68	75	57	18
Model 3 - ws varies	97	28	69	44	8	36	75	61	14
Model 4 - 0.1% air all pipes	97	37	60	48	17	31	74	67	7
Model 5 - 0.25% air all pipes	100	32	68	49	21	28	74	69	5
Model 6 - 0.5% air all pipes	98	43	55	48	26	22	74	70	4

Table 5.4 – Tabular Surge Results for Scenario #2 (continued)

Pump # (Time of Closure) Model Name	**Site #6**			**Site #5**			**Site #3**		
	Pressure Range		**Pressure Drop (psi)**	**Pressure Range**		**Pressure Drop (psi)**	**Pressure Range**		**Pressure Drop (psi)**
	Max (psi)	**Min**		**Max (psi)**	**Min**		**Max (psi)**	**Min**	
Pump 1 (30sec, 19 sec trip)									
Field	**100**	**40**	**60**	**47**	**35**	**12**	**77**	**70**	**7**
Model (AWWARF Study)	83	44	39	36	21	15	59	51	8
Model 1 - 3000fps all pipes	168	-14	182	50	-5	55	75	60	15
Model 2 - 3800 fps all pipes	168	-14	182	53	-12	65	75	53	22
Model 3 - ws varies	97	27	70	44	8	36	75	59	16
Model 4 - 0.1% air all pipes	97	37	60	48	16	32	74	68	6
Model 5 - 0.25% air all pipes	99	32	67	51	21	30	74	69	5
Model 6 - 0.5% air all pipes	100	43	57	49	26	23	74	70	4
Pump 1 (52sec, 33 sec trip)									
Field	**99**	**44**	**55**	**48**	**37**	**11**	**77**	**71**	**6**
Model (AWWARF Study)	84	50	34	36	23	13	59	53	6
Model 1 - 3000fps all pipes	205	-14	219	66	-14	80	75	56	19
Model 2 - 3800 fps all pipes	238	-14	252	58	-10	68	77	53	24
Model 3 - ws varies	99	27	72	45	8	37	75	59	16
Model 4 - 0.1% air all pipes	97	37	60	46	16	30	75	65	10
Model 5 - 0.25% air all pipes	103	32	71	48	21	27	75	69	6
Model 6 - 0.5% air all pipes	102	43	59	49	26	23	74	71	3

Table 5.5 – Difference between Model Results and Field Data for Scenario #2

Pump # (Time of Closure) Model Name	**Site #6**			**Site #5**			**Site #3**		
	Pressure Range (psi)		**Pressure Drop (psi)**	**Pressure Range (psi)**		**Pressure Drop (psi)**	**Pressure Range (psi)**		**Pressure Drop (psi)**
	Max	**Min**		**Max**	**Min**		**Max**	**Min**	
Pump 1 (22 sec, 16 sec trip)									
Field	0	0	0	0	0	0	0	0	0
Model (AWWARF Study)	-16	4	-20	-10	-14	4	-16	-18	2
Model 1 - 3000fps all pipes	55	-51	106	13	-43	56	1	-8	9
Model 2 - 3800 fps all pipes	20	-51	71	14	-40	54	1	-9	10
Model 3 - ws varies	-1	-8	7	-1	-23	22	1	-5	6
Model 4 - 0.1% air all pipes	-1	1	-2	0	-13	13	0	-1	1
Model 5 - 0.25% air all pipes	1	-5	6	-6	-12	6	0	1	-1
Model 6 - 0.5% air all pipes	1	6	-5	2	-7	9	0	2	-2
Pump 1 (24 sec, 16 sec trip)									
Field	0	0	0	0	0	0	0	0	0
Model (AWWARF Study)	-16	4	-20	-10	-14	4	-15	-17	2
Model 1 - 3000fps all pipes	64	-52	116	13	-44	57	2	-7	9
Model 2 - 3800 fps all pipes	15	-52	67	9	-48	57	2	-10	12
Model 3 - ws varies	-2	-10	8	-1	-26	25	2	-6	8
Model 4 - 0.1% air all pipes	-2	-1	-1	3	-17	20	1	0	1
Model 5 - 0.25% air all pipes	1	-6	7	4	-13	17	1	2	-1
Model 6 - 0.5% air all pipes	-1	5	-6	3	-8	11	1	3	-2

Table 5.5 – Difference between Model Results and Field Data for Scenario #2 (continued)

Pump # (Time of Closure) Model Name	**Site #6 Pressure Range Max (psi)**	**Site #6 Pressure Range Min (psi)**	**Site #6 Pressure Drop (psi)**	**Site #5 Pressure Range Max (psi)**	**Site #5 Pressure Range Min (psi)**	**Site #5 Pressure Drop (psi)**	**Site #3 Pressure Range Max (psi)**	**Site #3 Pressure Range Min (psi)**	**Site #3 Pressure Drop (psi)**
Pump 1 (30sec, 19 sec trip)									
Field	0	0	0	0	0	0	0	0	0
Model (AWWARF Study)	-17	4	-21	-11	-14	3	-18	-19	1
Model 1 - 3000fps all pipes	68	-54	122	3	-40	43	-2	-10	8
Model 2 - 3800 fps all pipes	68	-54	122	6	-47	53	-2	-17	15
Model 3 - ws varies	-3	-13	10	-3	-27	24	-2	-11	9
Model 4 - 0.1% air all pipes	-3	-3	0	1	-19	20	-3	-2	-1
Model 5 - 0.25% air all pipes	-1	-8	7	4	-14	18	-3	-1	-2
Model 6 - 0.5% air all pipes	0	3	-3	2	-9	11	-3	0	-3
Pump 1 (52sec, 33 sec trip)									
Field	0	0	0	0	0	0	0	0	0
Model (AWWARF Study)	-15	6	-21	-12	-14	2	-18	-18	0
Model 1 - 3000fps all pipes	106	-58	164	18	-51	69	-2	-15	13
Model 2 - 3800 fps all pipes	139	-58	197	10	-47	57	0	-18	18
Model 3 - ws varies	0	-17	17	-3	-29	26	-2	-12	10
Model 4 - 0.1% air all pipes	-2	-7	5	-2	-21	19	-2	-6	4
Model 5 - 0.25% air all pipes	4	-12	16	0	-16	16	-2	-2	0
Model 6 - 0.5% air all pipes	3	-1	4	1	-11	12	-3	0	-3

In review of Table 5.4 we find that the AWWARF models and the models run as part of this work both had trouble predicting the field data at each of the three sites. In terms of minimum pressure predictions, the AWWARF model was higher at site #6 (immediately downstream of tripped pumped) than the field data but was lower at the other sites for the remaining test. In contrast, the models as part of this work predicted lower values at all sites but predicted well at higher air entrainment values (i.e., lower celerity values)

Several methods were explored to determine which model fit the field test data the best. In the end, because each site or data point stands equally on its own a simple average of the percentage difference was computed and compared using equation 2.

Presented in Table 5.6 is a comparison of percent fit for each model run using the before mentioned best-fit approaches. From this data it can be seen that most of the model runs 4, 5 and 6 all yielded better fits than the AWWARF study in terms of the max. min. fit, except for the 52 second pump shutdowns on model 3. In each pump shutdown model 6 had the best-fit. This equates to a model with a celerity value that would have air entrainment of around 0.5%.

Also presented in Table 5.6 is the length of time in which pressures were below 20 psi for the best-fit models. As with the field results, the model did not predict any amount of time where the pressure at any of the sites was less than 20 psi.

Table 5.6 - Best-Fit Computation for Scenario #2 Surge Results

Pump # (Time of Closure) Model Name	**Max - Min Best Fit**	**Pressure Drop Best Fit**	**Time less than 20 psi(*) (sec)**
Pump 1 (22 sec, 16 sec trip)			
Model (AWWARF Study)	77%	82%	
Model 3 - ws varies	83%	144%	
Model 4 - 0.1% air all pipes	93%	115%	
Model 5 - 0.25% air all pipes	89%	114%	
Model 6 - 0.5% air all pipes	**92%**	**103%**	**0**
Pump 1 (24 sec, 16 sec trip)			
Model (AWWARF Study)	77%	82%	
Model 3 - ws varies	80%	153%	
Model 4 - 0.1% air all pipes	90%	126%	
Model 5 - 0.25% air all pipes	89%	129%	
Model 6 - 0.5% air all pipes	**92%**	**104%**	0
Pump 1 (30sec, 19 sec trip)			
Model (AWWARF Study)	77%	78%	
Model 3 - ws varies	77%	154%	
Model 4 - 0.1% air all pipes	88%	124%	
Model 5 - 0.25% air all pipes	88%	129%	
Model 6 - 0.5% air all pipes	**93%**	**106%**	0
Pump 1 (52sec, 33 sec trip)			
Model (AWWARF Study)	77%	74%	
Model 3 - ws varies	76%	174%	
Model 4 - 0.1% air all pipes	85%	139%	
Model 5 - 0.25% air all pipes	87%	144%	
Model 6 - 0.5% air all pipes	**93%**	**118%**	0

(*) at Site 5, actual field time below 20 psi was 0 seconds for each case.
value in **bold** is best-fit.

Presented in Figures 5.4 and 5.5 are graphs that illustrate how the best-fit data for the for 24 second and 52 second pump shutdown correlate with the field data. Each figure shows the field and model pressure at sites 3, 5 and 6.

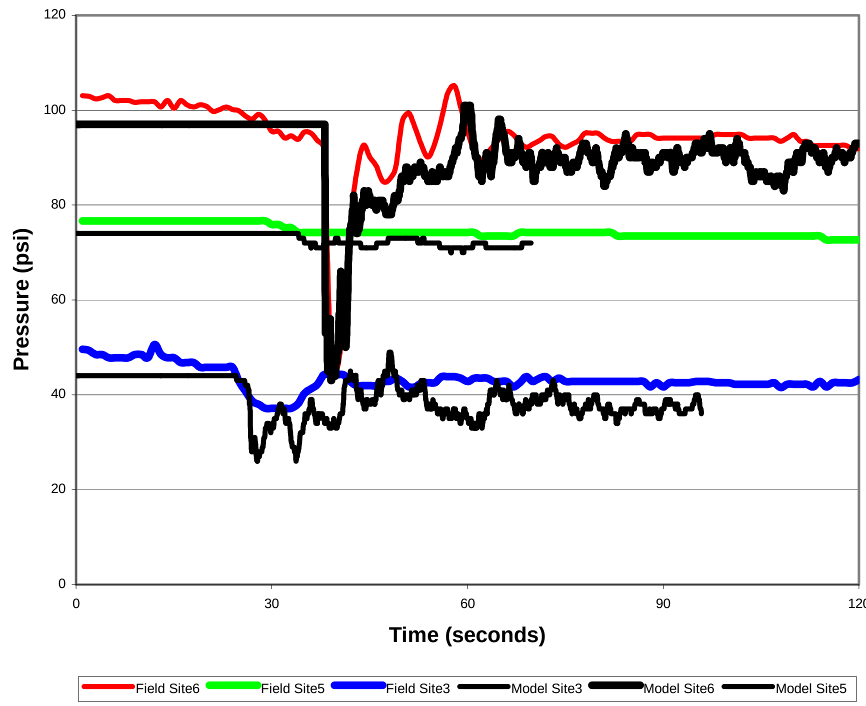


Figure 5.4 – Scenario #2: 52 second Shutdown Field vs. Model 6

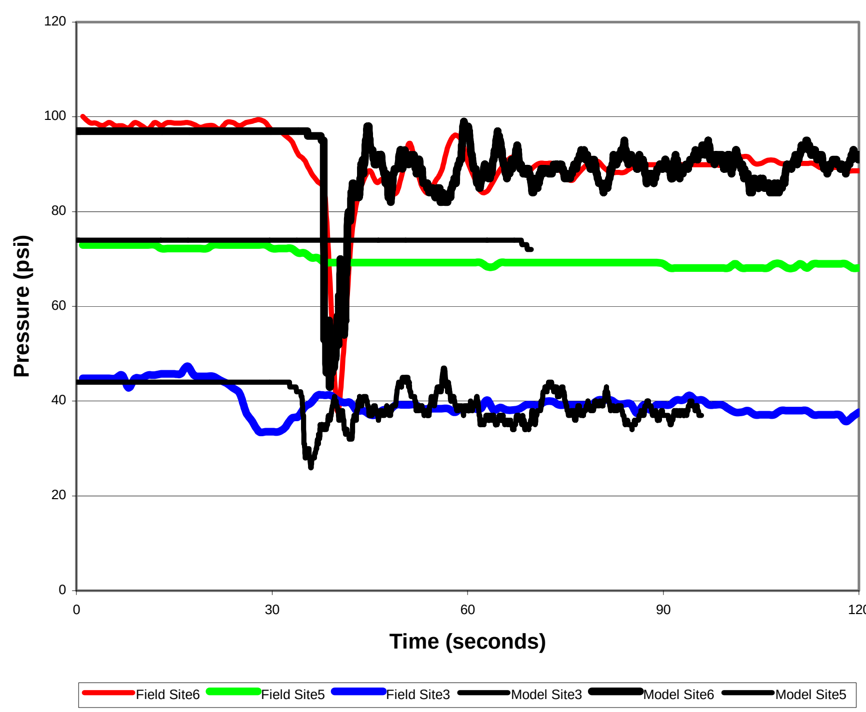


Figure 5.5 – Scenario #2: 24 second Shutdown Field vs. Model 6

5.4 Lightning Strike at Site 1 Results (Scenario #3)

The AWWARF study did not include this data set, which involved a lighting strike at WTP1 in the early morning hours of July 4, 2001. The plant was running near capacity (37 MGD) at the time of the lighting strike. Four high service pumps were running (one 700 Hp - 8 MGD pump, two 800 Hp – 8 MGD pumps and one 900 HP - 10 MGD pump). The surge wave was collected by field equipment located at Site #1 and #5.

Because the scenario involved a loss of power trip, the surge event was modeled as a pump trip at time equal to 1 second with a ball valve. Three of the four pumps tripped and shutdown along with their corresponding ball valve.

Table 5.7 - Tabular Surge Results for Scenario #3

Pump # (Time of Closure) Model Name	Site #1 Pressure Range Max (psi)	Site #1 Pressure Range Min (psi)	Site #1 Pressure Drop (psi)	Site #5 Pressure Range Max (psi)	Site #5 Pressure Range Min (psi)	Site #5 Pressure Drop (psi)
WTP1 Lightning Strike Field	157	14	143	29	0	29
Model 1 - 3000fps all pipes	147	6	141	24	-9	33
Model 2 - 3800 fps all pipes	519	-14	533	25	-14	39
Model 3 - ws varies	147	4	143	28	-6	34
Model 4 - 0.1% air all pipes	492	-14	506	24	-14	38
Model 5 - 0.25% air all pipes	436	-14	450	24	-14	38
Model 6 - 0.5% air all pipes	442	-14	456	24	-14	38

The lightning strike at WTP1 is somewhat difficult to model in the fact that the exact boundary conditions related to how the pumps shutoff is unknown. Did the ball valves close evenly, or did they remain stuck open for a period of time? Some information was gathered from the WTP1 operator who was working that night; however, certain assumptions had to be made in order to create the model. The assumptions made were that all valves closed per normal except for High service pump 11, which remained opened until the operator could close the discharge valve by hand. With that being said, in table 5.7 it is interesting to note that Model 2, 4, 5 & 6 did not correlate well at all. Models 1 and 3 however, performed well. Utilizing the best-fit "max-min" equation 2 approach, the computed best-fit value for model 1 and 3 in this scenario computed to only 73%. This is due to the percentage difference resulting from the large Site 5 minimum pressures result of 0 psi. Using the pressure drop magnitude best-fit approach, models 1 and 3 compute to 101% and 103%. To further illustrate how well the model correlated, presented in Figure 5.6 is a graph of the field data versus the best-fit model.

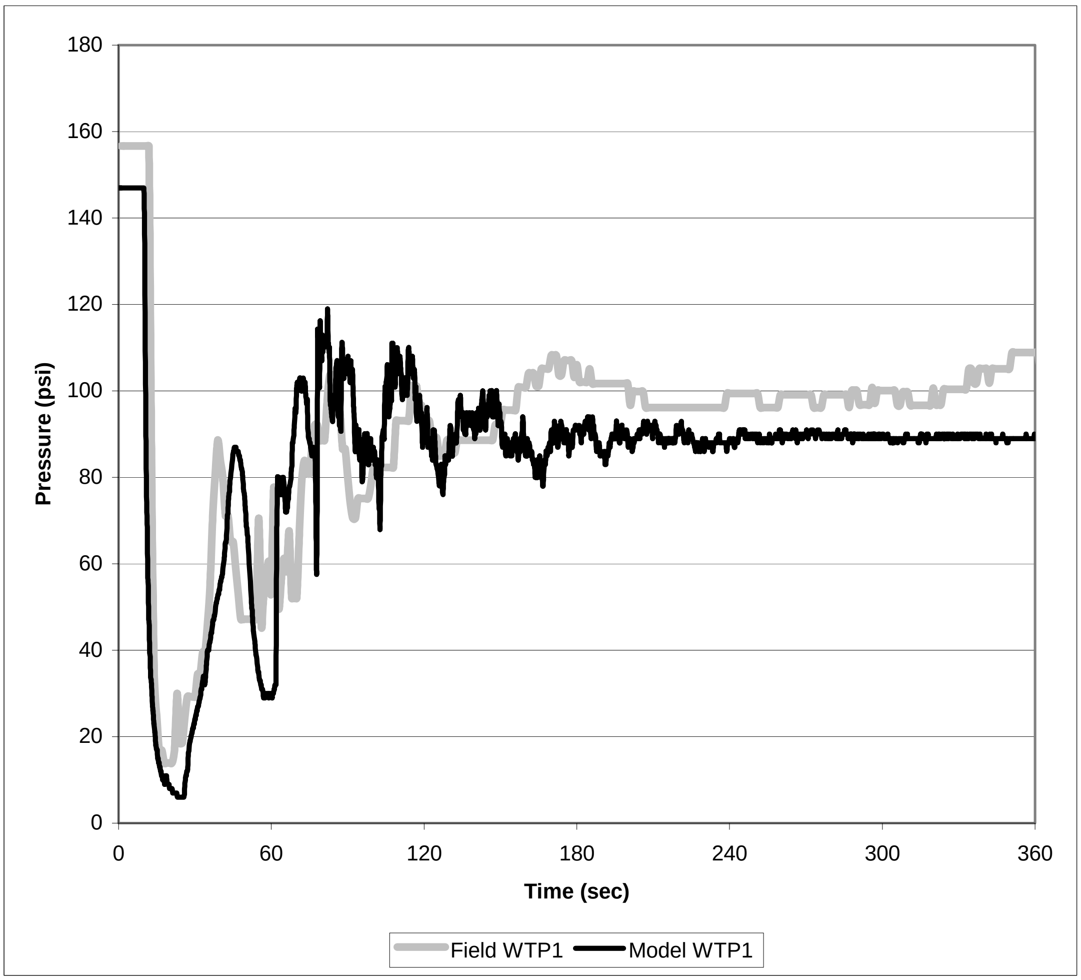


Figure 5.6 – Scenario #3: Field Data vs. Model at Site 1

From Figure 5.6 and 5.7 it can be seen that the model correlates very well with the field data both in terms of magnitude and general shape. Figure 5.7 differs slightly, but the key item is that is predicts low pressures events that could lead to potential distribution system intrusion. Because scenario #3 involved a real-life event, with no one present to record how things happened (i.e., actual speed of ball valve closure, did all pumps trip, etc) the ability to model after the event and have a model correlate well would increase one's confidence with the model.

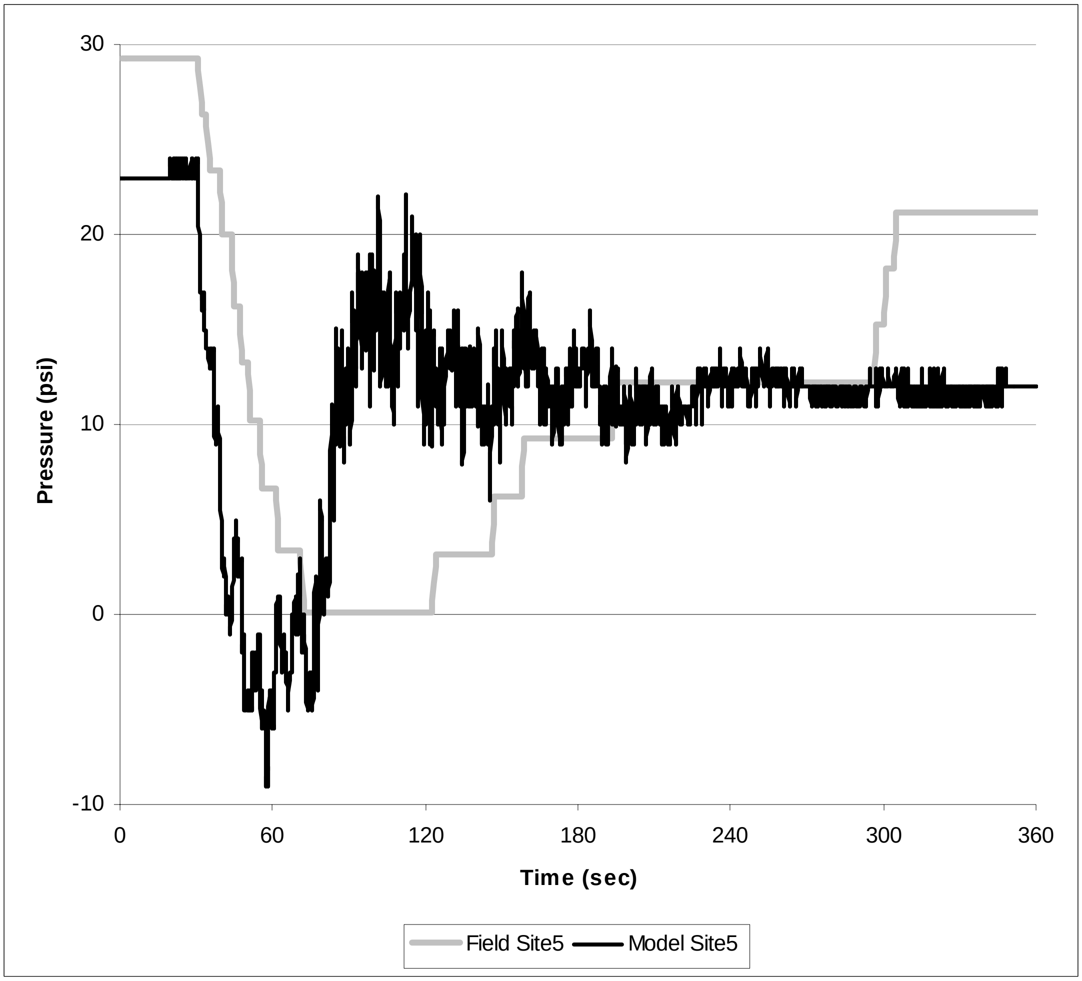


Figure 5.7 – Scenario #3: Field Data vs. Model at Site 5

The other important part from Figure 5.6 and 5.7 is the amount of time that pressure was below 20 psi. In Figure 5.6 the amount of time that pressure was below 20 psi in the field was 11 seconds versus the models computed time of 14 seconds. In Figure 5.7 the amount of time that pressure was below 20 psi was greater than 60 seconds for both the field and model.

5.5 Site #1 Drawdown Test Results (Scenario #4)

This test, also referred to as the Calibration Data test in the AWWARF Study involved four shutdowns and three startups of three different high service pumps at WTP1 (Site #1). The surge wave was collected by field equipment located at Sites #1, #2 and #3. The unique part of this test was that one of the two 30-inch transmission mains leaving the plant was valved off so that all the flow was proceeding through one 30-inch main. The one flaw in using this data as a calibration run is that SCADA data for the distribution system pressures and tank levels were not recorded for the date on which this drawdown test was conducted (3-15-01) due to a problem with the SCADA data logging software. As luck would have it the drawdown test conducted at WTP1 on 3-15-01 was also flawed and was repeated on 4-3-01. All SCADA data is available for that day and RADCOM recorders were set at Site #1, #2 and #3 for this test. This test will be used to compare model results versus actual results under the similar field conditions as the 3-15-01 date included in the AWWARF study. The drawdown involved three shutdowns and three startups of three different high service pumps at WTP1 (Site #1), with one of the shutdowns involving two pumps within a one minute time period. The surge wave was collected by field equipment located at Sites #1, #2 and #3. The pump startup was not modeled since they did not contribute to any low or negative pressures within the distribution system.

Presented in Tables 5.8 and 5.9 are the results of the modeling results for scenario #4. As has been the case with all prior modeling results, the results with 3800 fps wave speed were poor. In this case the results with 3000 fps wave speed and the variable wave speed by material type were also poor. The best model for the Pump 10 shutdown was using 0.5% air entrainment. The best-fit model for the pump 11 shutdown was using 0.1% air entrainment.

Table 5.8 - Tabular Surge Results for Scenario #4

Cause of Transient (Operating Condition)	Flow (Pre-condition) (MGD)		Site#	Field Measurements		Model 3000ws		Model 3800 ws		Model variable ws	
	Field	Model		Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)
Shutdown of HS Pump 10	8.8	9.3	1	124	70	125	161	125	630	125	196
			2	53	57	59	109	59	314	59	148
			3	45	44	55	100	55	249	55	122
Shutdown of HS Pump 11 & 14	15.8	16.3	1	161	140	160	708	160	548	160	236
			2	76	86	84	270	84	262	84	269
			3	61	69	75	260	75	224	75	140
Shutdown of HS Pump 11	8.5	9.0	1	138	130	131	661	131	601	131	197
			2	66	97	65	248	65	229	65	132
			3	56	59	62	203	62	218	62	117

Cause of Transient (Operating Condition)	Flow (Pre-condition) (MGD)		Site#	Field Measurements		Model 0.1% air		Model 0.25% air		Model 0.50% air	
	Field	Model		Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)
Shutdown of HS Pump 10	8.8	9.3	1	124	70	125	143	125	98	125	74
			2	53	57	59	101	59	82	59	62
			3	45	44	55	87	55	75	55	56
Shutdown of HS Pump 11 & 14	15.8	16.3	1	161	140	160	139	160	3500	160	1415
			2	76	86	84	106	84	98	84	451
			3	61	69	75	84	75	85	75	451
Shutdown of HS Pump 11	8.5	9.0	1	138	130	131	99	131	94	131	68
			2	66	97	65	90	65	80	65	57
			3	56	59	62	82	62	72	62	50

Table 5.9 - Difference between Model Results and Field Data for Scenario #4

Cause of Transient (Operating Condition)	Flow (Pre-condition) (MGD)		Site#	Model 3000ws		Model 3800 ws		Model variable ws	
	Field	Model		Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)
Shutdown of HS Pump 10	8.8	9.3	1	1	91	1	560	1	126
			2	6	52	6	257	6	91
			3	10	56	10	205	10	78
Shutdown of HS Pump 11 & 14	15.8	16.3	1	-1	568	-1	408	-1	96
			2	8	184	8	176	8	183
			3	14	191	14	155	14	71
Shutdown of HS Pump 11	8.5	9.0	1	-7	531	-7	471	-7	67
			2	-1	151	-1	132	-1	35
			3	6	144	6	159	6	58

Cause of Transient (Operating Condition)	Flow (Pre-condition) (MGD)		Site#	Model 0.1% air		Model 0.25% air		Model 0.50% air	
	Field	Model		Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)	Pre-Condition Pressure (psi)	Maximum Change in Pressure (psi)
Shutdown of HS Pump 10	8.8	9.3	1	1	73	1	28	1	4
			2	6	44	6	25	6	5
			3	10	43	10	31	10	12
Shutdown of HS Pump 11 & 14	15.8	16.3	1	-1	-1	-1	3360	-1	1275
			2	8	20	8	12	8	365
			3	14	15	14	16	14	382
Shutdown of HS Pump 11	8.5	9.0	1	-7	-31	-7	-36	-7	-62
			2	-1	-7	-1	-17	-1	-40
			3	6	23	6	13	6	-9

The best-fit model for the pump 11 and 14 concurrent pump shutdown was the 0.1% air entrainment model. Note that this shutdown modeling involved using two pump trips at different times and two valve closings at different times. Based upon the field data a pump trip of 17 seconds was used for pump 11 with valve closure starting at 1 second and completing at 26 seconds. Pump 14 was tripped at 61 seconds with valve closure starting at 46 seconds and completing at 86 seconds.

Presented in Figure 5.8 is a graph of the field data for the surge event modeled in Scenario #4 at Site #1, shutdown of pumps 11 & 14 versus the modeling data for the best-fit model (0.1% air entrainment). Note the good correlation between the low pressure magnitude created when pump 14 shuts down. Also note how well the slopes of model correlate with the field data.

Presented in Figure 5.9 is a graph of the field data for the surge event modeled in Scenario #4 at Site 2, shutdown of pumps 11 & 14 versus the modeling data for the best-fit model (0.1% air entrainment). There are several things to note in this figure. The first being that the elevation or starting pressures differ by 13 psi. This appears problematic; however, the elevation of site used was never surveyed and was taken from USGS Quad sheet as elevation 1046. The elevation in the model for this node is 1040. QUAD sheets use 10 foot contour interval and may be off as much as 20 feet. Considering this, the starting pressure difference is reasonable. The next difference is the amount of time difference between the peaks. This is accounted for in the fact that there was no correlation between the clocks used on the field data loggers used at site 1 and 2. Because the units were unable to be set with the same zero start time there is some built in difference. The model results, in terms of time are in proper relation because the zero start time is the same. The important items to note are the magnitudes and slopes and in Figure 5.9 the model correlates very well with the field data. The model predicted negative pressures that were also recorded by the data loggers.

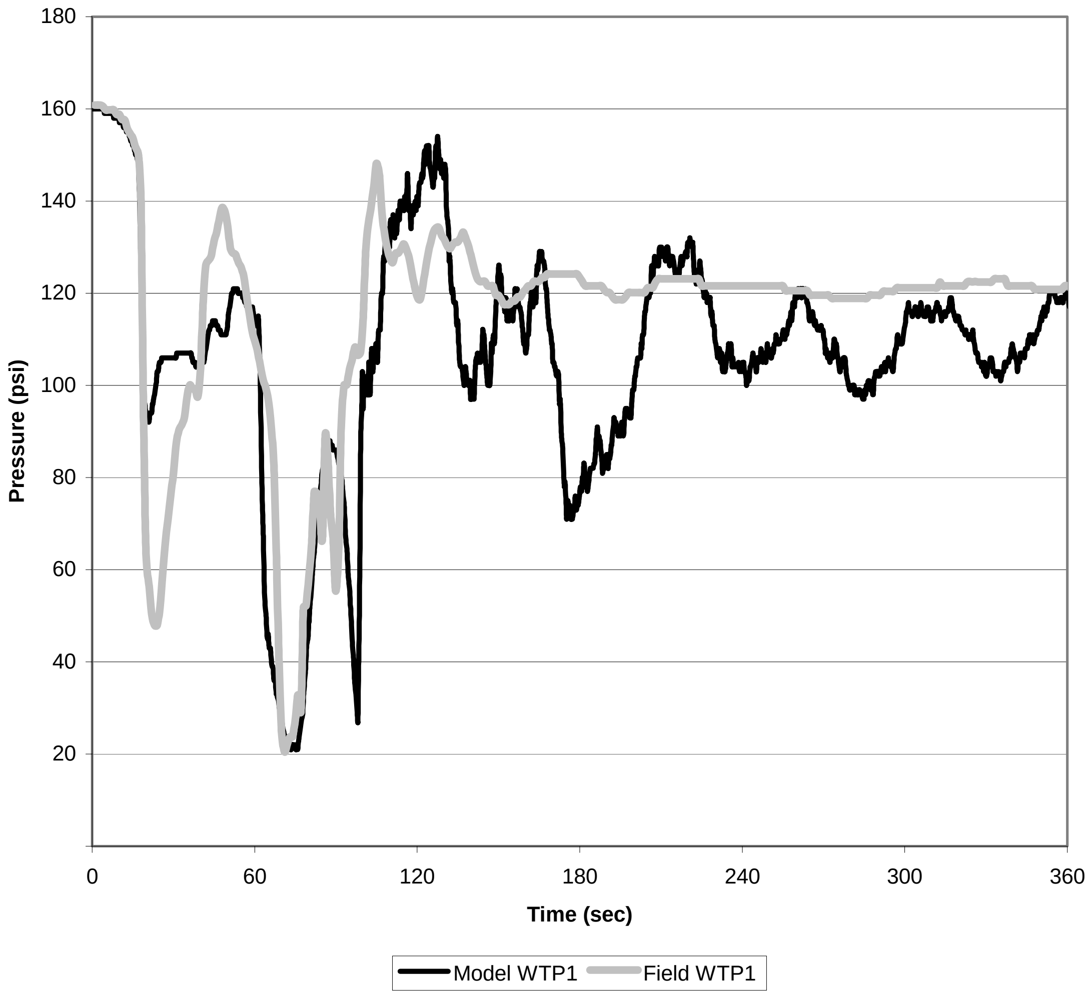


Figure 5.8 – Scenario #4: Field Data vs. Model at Site 1
Shutdown of Pumps 11 & 14

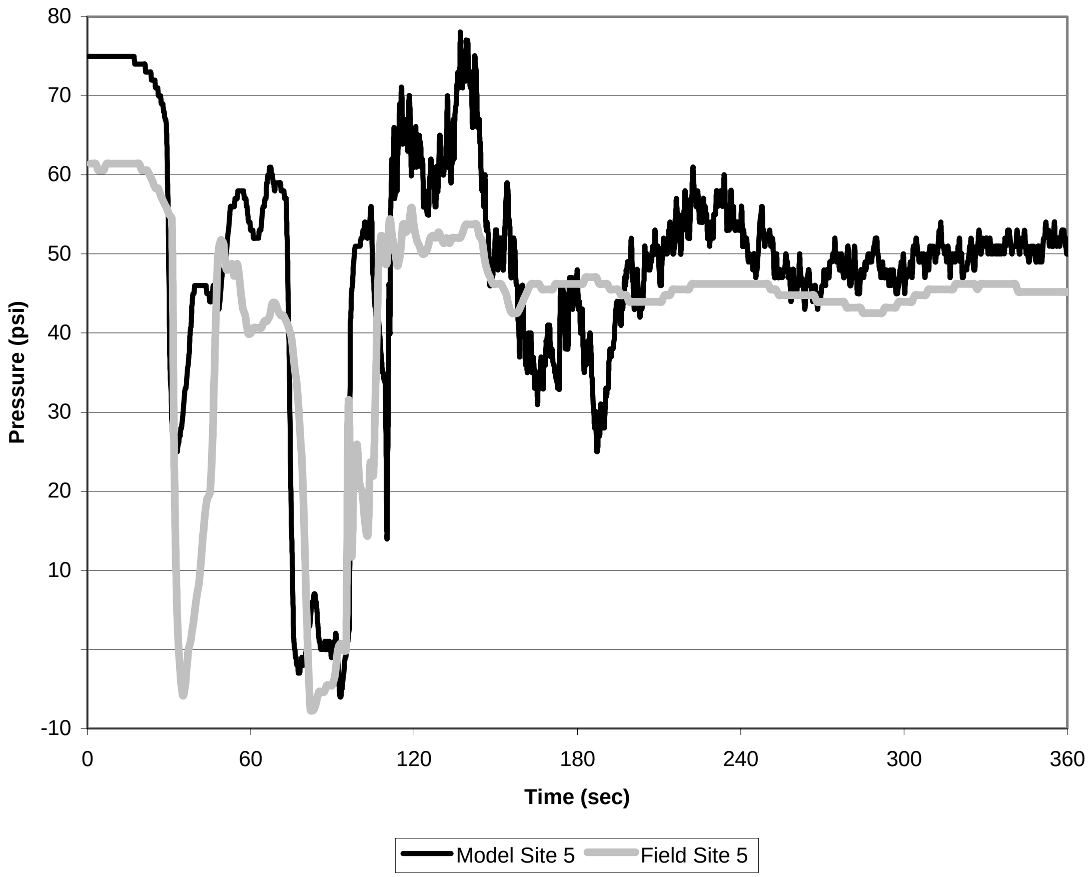


Figure 5.9 – Scenario #4: Field Data vs. Model at Site 2
Shutdown of Pumps 11 & 14

The other important part from Figures 5.8 and 5.9 is the amount of time that pressure was below 20 psi. In Figure 5.8 neither the model nor field data had pressures below 20 psi. In Figure 5.9 the amount of time that pressure was below 20 psi in the field was 31 seconds versus the models computed time of 22 seconds.

5.6 Locations of Low or Negative Pressures

As discussed in section 4.9, the use of the pressure contour tool within Surge2000 provides engineers with a powerful tool to locate places within a distribution system where low pressure below are likely to occur.

Presented in Figure 5.10 is a figure of the entire surge model, (same base as in Figure 4.17) with the minimum pressures computed. The area with gray cross hatched contours represents areas that experienced minimum pressure below 20 psi.

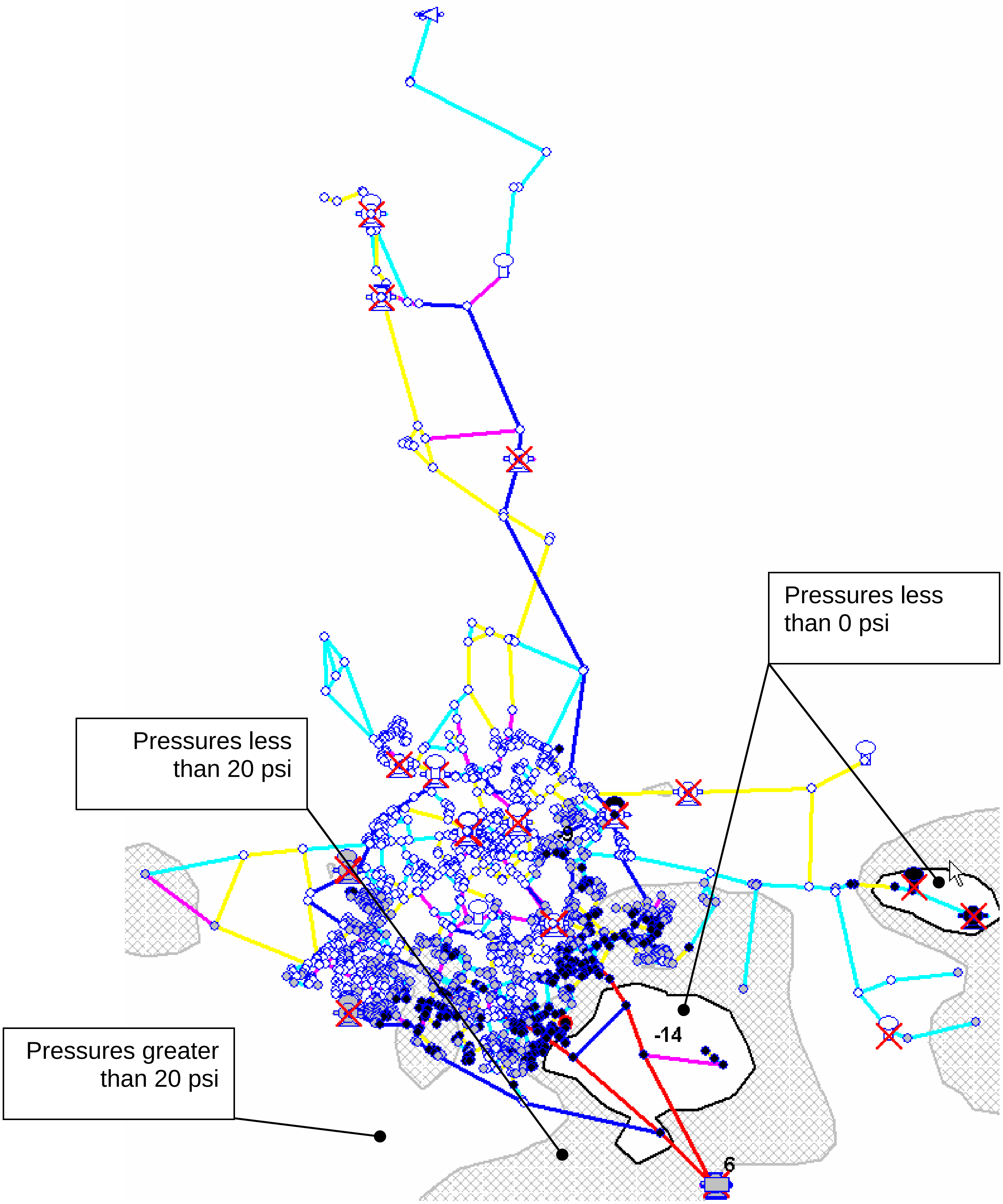


Figure 5.10 – Locations of Minimum Pressures below 20 psi.

The areas in all white experienced pressures greater then 20 psi. The white areas bordered by black lines experienced pressures below 0 psi. These are the areas that would warrant further review to determine the risk potential for contamination. High hazard areas would be where leaking mains are located near underground storage tanks or near sewer lines. Areas located in these potentially higher risks areas should be fixed quickly and placed on a priority list for fixing.

5.7 Effects of Low or Negative Pressures

It has been documented that low pressures do exist within water distribution systems. In order to help understand the effects of low pressures within a distribution system consider two examples presented herein. Example 1 is a leaking water main located within 18 inches of a leaking surcharged sewer. A surcharged sewer is defined as a sewer that is under pressure, but not overflowing out of a manhole, due to heavy flow, line blockage or line size that is under capacity. Example 2 is a 2 story residential house with standard plumbing located within 50 feet of the location of a negative pressure event.

For simplicity, lets assume the leaking water main is under 35 psi and produces a 1-gallon per minute leak. Using the standard orifice equation one can determine the area of the leak to be approximately 0.08 inches (5/64) in diameter. Typical water lines are 4 feet below grade and lets assume that the sewer is 5.5 feet below grade but due to its surcharge condition is under 1 psi of pressure. Based on the results in scenario #3, site 5, zero psi was observed and occurred for approximately 50 seconds. Under this scenario, the head available to push water into the main is 0.8 feet (1 psi – 18 inches). The flow entering the system is 0.1 gallon per minute. Since the leak occurred for almost one minute 0.1 gallons of raw sewage could enter the water main. Could this amount make some one sick? That question is beyond the scope of this work, such factors as dilution, chlorine demand, etc. will determine what happens to that 0.1 gallons. However, it illustrates the point that water that is outside the main can potentially enter the main due to low pressures caused by a transient event . If the main was under a negative pressure of –14 psi then the amount would be approximately 0.6 gallons.

For example 2 lets consider a residential connection that is 50 feet in length from the water main to the house, and lets assume that a toilet on the second floor contains a blue tinted toilet cleaning product. Now lets assume that during the time when the toilet was filling, a negative pressure wave of 0 psi occurred for 50 seconds at the service connection. The head available to push flow from the toilet tank towards the main (assumes main elevation is 15 feet lower than tanks) is 15 feet. Assuming 75 feet of ¾ inch material the flow computed using a Hazen-Williams ‘C’ factor of 140 is 7.7 gpm. Considering that 75 feet of ¾ line contains 1.75 gallons of water it is clear that flow from the toilet tank could reverse all the way into the distribution system.

During the several models run it was shown that low and or negative pressures occurred in the distribution system from normal events and that pressures below 20 psi occurred for as long as 60 seconds. Given the two examples it is reasonable to assume that cross connections can occur within a water distribution systems that are subject to transients.

5.8 Methods to Reduce or Eliminate Low or Negative Pressure

The purpose of the work is not to discuss how negative pressures can be avoided within a water distribution system. Friedman, et. al. (2004) and Gullick, et. al. (2004) lists several methods such as slower valve closing speeds, air / vacuum valves, pressure surge vessels, surge relief valve, surge anticipator valves, etc. This work was conducted to show how the surge model can be used to verify and predict where low and or negative pressures occur. Furthermore, once a surge model is calibrated, solutions can be designed and modeled to determine the adequacy of the proposed solution. If a surge model can successfully predict low pressures, then the surge model becomes an extremely effective tool to design improvements to reduce and or eliminate the low pressure occurrences.

5.9 Discussion of Current Regulation Regarding Pressure

There are several guidelines and regulations that come into play when discussing low pressures. Often considered the "Bible" for water distribution systems, Recommended Standards for Water Works, 2003 Ed. (a.k.a. Ten State Standards), a publication printed by the Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers states the following concerning pressures (Chapter 8.2.1):

> "All water mains, including those not designed to provide fire protection, shall be sized after a hydraulic analysis based on flow demands and pressure requirements. The system shall be designed to maintain a minimum pressure of 20 psi (140 kPa) at ground level at all points in the distribution system under all conditions of flow. The normal working pressure in the distribution system should be approximately 60 to 80 psi (410 – 550 kPa) and not less than 35 psi (240 kPa)."

In the Commonwealth of Kentucky, the Environmental Protection Cabinet, Division of Water (KYDOW), governs water distribution systems. KYDOW publishes and enforces the rules and regulations of the Commonwealth. There is no law that directly mandates minimum pressures in Kentucky; however 401 KAR 8:100 Section 4 (1) references and incorporates the above Ten State Standards publication. Thus 20 psi is the minimum allowable pressure under all flow conditions in Kentucky.

In addition to the KYDOW, the Kentucky Public Service Commission (KYPSC) has jurisdiction and laws that apply to certain types of private water utilities and water districts within the Commonwealth. Municipal governments in Kentucky are not governed by these laws. 807 KAR 5:066, states "…In no event, however, shall the pressure at the customer's service pipe under normal conditions fall below thirty (30) psig nor shall the static pressure exceed 150 psig."

What is interesting to note is that Ten State Standards (TSS) mentions that 20 psi must be maintained under all flow conditions. Does this include transient events? If it does, then the state of Kentucky, by referencing TSS would consider it a violation of its regulation every time pressure dropped below 20 psi within a water distribution system.

Looking at the KYPSC regulation it clearly states that 30 psi must be met under normal conditions. Because transients occur infrequently, under this rule there would not be a violation during a transient event.

How does this all relate to this work? Perhaps this is ultimately a questions for attorneys, however, it seems reasonable that there is potential in the future, considering the nature of litigation in the United States, to sue a water utility should someone become sick and it is shown that the affected person lives in an area susceptible to pressure below 20 psi or susceptible to transient induced negative pressures. The surge model is a tool that can perform a couple of task in this regard. One, it can determine which areas are likely to see pressures below 20 psi as a result of a pump trip or rapid valve closure and two, it can be used to design systems to eliminate the effects of rapid valve closure or pump trip.

5.10 Application of Results for New Studies

One of the challenges of creating any surge model is what initial values should be used for newly created models and what values should be used when field data is not available. Based on this work, recommended values for celerity or wave speed should be either 3000 fps or based on pipe type and material with 0.1% air entrainment. The 3000 fps value was the best-fit model in a majority of the scenarios and is the best place to start from, while models with 0.1% air also correlated well. Models with 0.1% air had in general, wave speeds between 1800 – 2100 fps. The advantage of using 3000 fps is that the input of data will be fairly easy in that all pipes will have the same value. The one draw back to using this constant value is that it represents a system with a vast majority of pipe being rigid (concrete, AC, DI or CI). If the system is comprised of a large quantity of PVC or HDPE mains than 3000 fps will be to conservative.

As a final recommendation, for systems with unknown pipe material, 3000 fps is the best starting place for wave speed values. For models in which the materials are known, the use of a wave speed based on material type should be utilized as the starting point and then factored to account for air entrainment.

This latter approach will account for system that have both rigid and less rigid pipe and should yield better results.

CHAPTER 6
CONCLUSIONS

In Section 3.2, the results of the AWWARF project 2686 for system #2 were presented. This study was undertaken to determine if the conclusions that were written regarding system #2 in the AWWARF / EPA publication titled Verification and Control of Pressure Transients and Intrusion in Distribution System, page xxiv, were in fact true. This study was also undertaken to determine if large, complex water distribution system could be modeled effectively for transient events given the complexities arising out of demands, boundary conditions and appropriate celerity values.

The Environmental Protection Agency (EPA) has generally defined a large water system to be a system that supplies water to a population base greater than 100,000 people. This definition has been used a number of times when new regulation are to be promulgated and was used as the basis for defining water system under the requirements of the Public Health Security and Bioterrorism Preparedness and Response Act (PL. 107-188) of 2002. System #2 serves a population of approximately 350,000 people and thus is defined as a large water distribution system.

Based on the four different scenarios investigated and the multiple model runs per scenario, a total of sixty-six models were run and analyzed as part of this study. This study showed that the Surge2000 surge model was able to obtain better correlation than the modeling done as part of the AWWARF study. The reasons for this are listed below:

- EPS runs were conducted on the base models prior to performing surge models. This step insured that demand factors, pump and pipe status and tank levels were correct. It also was used to verify that the model held up or remained stable under all actual conditions.
- Demand factors were computed for every hour in order to get an accurate demand included with the model.
- Pump and tank changes were included with the modeling in order to get the steady state pressures to correlate between the model and field data.

- Several large users have the ability to greatly affect system pressures. The demands of the large users were evaluated to verify their effect during the surge events.
- Different values of celerity were used to determine which set of values correlated the best. The AWWARF study did not mention how it handled different celerity values.
- The pump trip was modeled accurately. Because all the pumps tested featured ball check valves, the shutdown of the pump had to be modeled as two events. One ball valve closing event and one pump trip during the ball valve closing event. The AWWARF study did not mention how it modeled the pump shutdown; however, if it simply based the shutdown on the ball valve closing time their model would not be reflecting true conditions.

Further, the Surge2000 model tended to correlate well with the field data in terms of magnitude, length of time for low or negative pressures and predicted slightly more conservative values then the actual field results. This slight conservativeness lends itself well to how transient models will be used by practicing engineers. If the engineering profession designs a solution to a problem using an effective transient model, there stands an excellent chance that the solution will truly work. An example of this is discussed below. Suppose an area of a water distribution system is located on high ground near a gas station that has two leaking diesel fuel tanks. Even though no one may have ever complained about funny tasting water, with an effective transient model, an analysis could be run to determine if negative pressure are ever likely to occur in the area.

From the results of this study and as mentioned previously, the model tended to be more conservative. Thus in the case of the leaking fuel tanks, if the model predicted pressures of 20 psi, it is likely that the actual field data would be higher and there would be very little risk of a cross connection. Likewise if the model predicted –14 psi, there stands an excellent chance that a cross connection would exist. Even with a slightly conservative transient model, proposed solutions or modifications that are made to reduce the low pressure event, there is an excellent chance that the solution would be effective in removing the cross connection.

The unique part of this project had to do with the size of the transient model and the ability to correlate transient events with real field data. This worked showed that large transient models, modeling large water distribution system, can effectively determine the area or location, length of time and magnitude of transient events. Based on the author's knowledge and research, no other published work exists, other than the referenced AWWARF study and citied papers, which documents this type transient modeling effort and the results that were obtained. Presented in Table 6.1 and 6.2 is a summary of the results of this study at Site 5 which is the highest ground elevation point in the system 2 distribution system.

Table 6.1 – Comparison of Max – Min Results at Site 5

Scenario#	**Field Data Max – Min (psi)**	**AWWARF Study Max – Min (psi)**	**This Study (*) Max – Min (psi)**
Scenario 1 – Pump 14 Shutdown	40 - 33	30 – 23	41 - 25
Scenario 1 – Pump 10 Shutdown	39 - 32	29 – 24	41 - 27
Scenario 1 – Pump 11 Shutdown	40 - 32	29 – 16	43 - 33
Scenario 2 – 22 sec close	45 - 33	35 – 19	47 - 26
Scenario 2 – 24 sec close	45 - 34	35 – 20	48 - 26
Scenario 2 – 30 sec close	47 - 35	36 – 21	49 - 26
Scenario 2 – 52 sec close	48 - 37	36 – 23	49 - 26
Scenario 3 – WTP1 Lightning Strike	29 - 0	n/a	28 - (-6)

(*) Using best-fit model.

Table 6.2 – Comparison of Length of Time below 20 psi at Site 5

Scenario#	Field Data Time less than 20 psi (sec)	AWWARF Study Time less than 20 psi (sec)	This Study (*) Time less than 20 psi (sec)
Scenario 1 – Pump 14 Shutdown	0	n/a	0
Scenario 1 – Pump 10 Shutdown	0	n/a	0
Scenario 1 – Pump 11 Shutdown	0	n/a	0
Scenario 2 – 22 sec close	0	n/a	0
Scenario 2 – 24 sec close	0	n/a	0
Scenario 2 – 30 sec close	0	n/a	0
Scenario 2 – 52 sec close	0	n/a	0
Scenario 3 – WTP1 Lightning Strike	11	n/a	14

(*) Using best-fit model.

Site 5 is located near industrial and commercial developments and generally is the most pressure sensitive portion of the system 2 distribution system. While other sites such as Sites 2 and 3 had low and or negative pressures during pump shutdowns or trips the other areas were not located in highly developed areas subject to sewers, gas stations and numerous service connections.

In all the model runs and created as part of the work, it became apparent that the value of celerity is the most important factor in determining the magnitude and duration of a low and or negative transient event. In some models, the best or optimum results were somewhere between a fixed value of 3000 fps for all pipes and a celerity value based on pipe material, size and percentage of entrained air. Because of the difficulty in knowing which pipes would have entrained air and which pipes would not, the models run as part of this study were all considered to have the same amount of entrained air. In real life, one would not expect that to be the case, but in the final analysis, the large models were able to correlate well with the field data using this approach.

Based on this work, it is recommended that wave speed values of 3000 fps be used for newly created surge models in which field data to calibrate is not available. This value indicates that the system is comprised of mostly rigid pipe with air entrainment. If it is known that a system is comprised of mostly PVC and HDPE pipe than the 3000 fps value is to high. If material type data is available for all pipes within the model, it is further recommended that a second model be created using wave speeds based on material type and 0.1% air entrainment. This will result in a model with wave speeds between 1800 and 2100 fps and should work regardless of pipe material makeup of the distribution system.

Skeletonization of models is known to effect the correlation between field data and model results. This work, which included a 2,500 pipe model that represented approximately 40% of the entire distribution system correlated well. it is expected that smaller models with less percentage of mains modeled would yield different results although this assumption was not tested within this work.

CHAPTER 7
FUTURE RESEARCH

Future research can be construed to mean a couple of different items. In one context it can be interpreted to mean areas of future research needed to help understand items discovered or un-quantified during a research project, while in another context it can mean how the current project could be continued to provide further insight in the future.

This work can benefit from three areas going forward. The first area of study would be the continuation of work as it relates to air entrainment within a water distribution system. The models used in this study were indeed sensitive to the amount of air entrainment. If field loggers were not available there would be no true way of knowing what the appropriate celerity values should be. Efforts were made to look at the effect of air entrainment by running multiple model runs with different celerity values.

The work can also benefit from further study and modeling in the future after changes to the distribution system are installed. One example of this is that by the beginning of 2006 a new floating 2 million gallon elevated storage tank will be constructed very near Site #5. This new tank affords the opportunity to truly study a system as it grows and allows for before and after comparisons. As discussed in the review of the work conducted in Austin, Texas, it is assumed that online floating storage tanks can reduce low pressure transients. This assumption could be tested upon the completion of this new 2 MG floating storage tanks.

The final area of future study would be the effect of skeletonization on this surge model. Currently there exist three models for System #2. A 12,500 pipe model, a 5,500 pipe model and the 2,500 pipe model used as part of this work. Re-running these scenarios on these larger models, as well as running these on smaller models (as low as 10%) would be valuable information to engineers and modelers in that it would help to determine what size model is the most appropriate in terms of model accuracy versus cost to develop.

APPENDICES

APPENDIX A

BASE SURGE MODEL INPUT FILE

This Appendix was to originally include a hard copy of the base surge model used to create the surge models in this work; however, due to security issues relating to the protection of public drinking water systems the base surge model is not included. Researchers may request a copy of the base surge model upon a background check and security release from the Author and the American Water Works Company. The author can be reached at rcsvin@aol.com or rcsvin@amwater.com.

APPENDIX B

BOUNDARY CONDITIONS & PUMP INERTIA

This Appendix includes a sample of the boundary condition data and pump inertia calculations that were used in the surge models.

Water Treatment Plant Pump Logs

Site #9 and Site #1 Pump Log for Oct. 18, 2002			
	Shift		
Pump	12am - 8am	8am - 4pm	4pm - 12am
Site 9 - WTP2			
WTP2 - #6			
WTP2 - #7	On	On	On
WTP2 - #8			
WTP2 - #9			
WTP2 - #10			
WTP2 - #11			
Site 1 - WTP1			
WTP1 - #10		On 8:55	Off 4:45
WTP1 - #11	On	On	On
WTP1 - #12			
WTP1 - #13			
WTP1 - #14	On	On	On
WTP1 - #15	On	On	On

Site #9 and Site #1 Pump Log for Oct. 15, 2002			
	Shift		
Pump	12am - 8am	8am - 4pm	4pm - 12am
Site 9 - WTP2			
WTP2 - #6			
WTP2 - #7	On	On	On
WTP2 - #8			
WTP2 - #9			
WTP2 - #10			
WTP2 - #11			
Site 1 - WTP1			
WTP1 - #10		On 1:46, Off 1:59	
WTP1 - #11		On 2:06, Off 2:26	
WTP1 - #12			
WTP1 - #13	On		On
WTP1 - #14	On	Off 1:43, On 2:46	On
WTP1 - #15	On		On

Site #9 and Site #1 Pump Log for July 4, 2001			
	Shift		
Pump	**12am - 8am**	**8am - 4pm**	**4pm - 12am**
Site 9 - WTP2			On 8:00
WTP2 - #6			
WTP2 - #7	On	On	Off 8:00
WTP2 - #8			
WTP2 - #9			
WTP2 - #10			
WTP2 - #11			
Site 1 - WTP1			
WTP1 - #10	Off 1:38, On 2:50	On	Off 4:45
WTP1 - #11	On	On	On
WTP1 - #12	Off 1:38		
WTP1 - #13	Off 1:38, On 3:05	On	On
WTP1 - #14	On 2:55	On	On
WTP1 - #15			

Site #9 and Site #1 Pump Log for April 3, 2001			
	Shift		
Pump	**12am - 8am**	**8am - 4pm**	**4pm - 12am**
Site 9 - WTP2			
WTP2 - #6	On 1:00	On	On
WTP2 - #7	On	On	On
WTP2 - #8			
WTP2 - #9			
WTP2 - #10			
WTP2 - #11			
Site 1 - WTP1			
WTP1 - #10	On	Off 8:30	On 11:00
WTP1 - #11	On	Off 8:25, On 9-9:55, 11:05-12:05, On 1:40	
WTP1 - #12			
WTP1 - #13			
WTP1 - #14	On	Off 8:20, On 9-9:55, On 12:55	
WTP1 - #15			

Booster Station Pump Log

Booster Pump Log for October 18, 2002			
	Shift		
Site #	12am - 8am	8am - 4pm	4pm - 12am
14			On 4:55, Off 7:50
14a			On 8:20, Off 9:15
15		9:30 - 10:00, 10:30 - 11:00	On 7:10, Off 8:20
4			On 7:50, Off 9:15
16			On 9:15, Off 11:00
6a	On 7:00 - 8:20	Ball Valve Testing	
6b		Ball Valve Testing	
6c		Ball Valve Testing	
17a			
17b			
19a			
19b			
19c			
20a			
20b			
21a			
21b			
13a	On 6:30	Off 9:50	
13b			
22	in auto control for (On 29’ – 35’)		
18	In auto Control (off all day)		

Booster Pump Log for October 15, 2002			
	Shift		
Site #	12am - 8am	8am - 4pm	4pm - 12am
14		On 2:30	Off 5:00
14a	On 6:15	Off 9:30	
15			On 5:00 - 7:20
4			On 6:20 - 10:20
16			On 7:30 - 11:00
6a		On 11:20 to 1:20	
6b			
6c			
17a		On 12:30	On
17b			
19a			
19b			
19c			
20a			
20b			
21a			

21b			
13a	On 6:20	Off 11:00	
13b		On 1:20	Off 5:00
22	in auto Control for (on 28' off 35')		
18	In auto Control (on 7:00 AM to 5:00 PM)		

Booster Pump Log for July 4, 2001			
	Shift		
Site #	12am - 8am	8am - 4pm	4pm - 12am
14	On 1:40-2:50		On 9:15-11:38
14a			On 4:30-9:15
15			On 9:15-9:45, On 10:45-11:40
4			
16		On 9:50-2:10	
6a	On 1:40-2:55	On 10:15-1:05	
6b	On 1:45-2:40		
6c			
17a			
17b			
19a			
19b			
19c			
20a			
20b			
21a			
21b			
13a			On 4:30-9:15
13b			
22	in auto Control		
18	In auto Control		

Booster Pump Log for April 3, 2001			
	Shift		
Site #	12am - 8am	8am - 4pm	4pm - 12am
14			
14a		On 9:15-3:10	On 8:30-9:40
15		On 9:55	Off 4:00, On 6:20-8:30, On 10:30-11:00
4			
16			
6a		On 8:20-12:55	
6b			
6c		On 8:30-9, On 9:55-12:55	On 9:40-11:00
17a			
17b			
19a			

19b			
19c			
20a			
20b			
21a			
21b			
13a	On 7:10-7:40	On 8:10-9:00, On 9:55-1:40	
13b			On 5:15-11:00
22	in auto Control		
18	In auto Control		

Base Model Changes for 10-18-02

Pipe	Time			Comment
10926	2	O	Open	CoxE Feed line
10926	3.25	C	Close	CoxE Feed line
12364	0	O	Open	York Feed Line
12364	3.5	C	Close	York Feed Line
12375	0	O	Open	Mercer Feed Line
12375	3	C	Close	Mercer Feed Line
12417	5.25	O	Open	Parkers Mill Feed Line
12417	8	C	Close	Parkers Mill Feed Line
12317	3.3	O	Open	CoxG Feed Line
12317	3.7	C	Close	CoxG Feed Line
12317	4.5	O	Open	CoxG Feed Line
12317	5.5	C	Close	CoxG Feed Line
12389	3.3	O	Open	Hume Feed Line
12389	4.5	C	Close	Hume Feed Line
12421	0	O	Open	Clays Mill Feed
12421	3.3	C	Close	Clays Mill Feed
12375	22	O	Open	Mercer Feed Line
12364	22.5	O	Open	York Feed Line

Site #	Time			Comment
WTP2-7	0	O	Open	Running all day
WTP1-10	9	O	Open	Pump Start
WTP1-10	16.75	C	Close	Pump Shutdown
WTP1-11	0	O	Open	Running all day
WTP1-14	0	O	Open	Running all day
WTP1-15	0	O	Open	Running all day
14 Booster	17	O	Open	
14 Booster	19.8	C	Close	
14a Booster	20.3	O	Open	
14a Booster	21.25	C	Close	
15 Booster	19.16	O	Open	
15 Booster	20.3	C	Close	
4 Booster	19.8	O	Open	
4 Booster	21.25	C	Close	
16 Booster	21.25	O	Open	
16 Booster	23	C	Close	
13 #1	6.5	O	Open	
13 #1	9.8	C	Close	
6a	7	O	Open	
6a	8.3	C	Close	
6c	14.58	O	Open	Ball Valve testing
6c	14.72	C	Close	Ball Valve testing
6c	14.92	O	Open	Ball Valve testing
6c	15.1	C	Close	Ball Valve testing
6c	15.36	O	Open	Ball Valve testing
6c	15.53	C	Close	Ball Valve testing
6c	15.75	O	Open	Ball Valve testing
6c	15.93	C	Close	Ball Valve testing

Base Model Changes for 7-4-01

Pipe	Time			Comment
10926	0	O	Open	
10926	1.75	C	Close	
12364	0	O	Open	
12364	5.5	C	Close	
12375	0	O	Open	
12375	1.5	C	Close	
12375	3	O	Open	
12375	5.5	C	Close	
12375	7.75	O	Open	
12375	8.75	C	Close	
12417	3	O	Open	
12417	7	C	Close	
12421	0	O	Open	
12421	1.75	C	Close	
12421	3	O	Open	
12421	7.5	C	Close	

Node	Time			Comment
WTP1-10	1.75	C	Close	
WTP1-10	3	O	Open	
WTP1-12	1.75	C	Close	
WTP1-13	1.75	C	Close	
WTP1-13	3	O	Open	
WTP1-14	3	O	Open	
14 Booster	1.75	O	Open	
14 Booster	2	C	Close	
14 Booster	21.25	O	Open	
14 Booster	23.5	C	Close	
14A Booster	16.5	O	Open	
14A Booster	23.5	C	Close	
15 Booster	21.25	O	Open	
15 Booster	21.75	C	Close	
15 Booster	22.75	O	Open	
15 Booster	23.75	C	Close	
16 Booster	9.75	O	Open	
16 Booster	2.25	C	Close	
6A	1.75	O	Open	
6A	3	C	Close	
6B	1.75	O	Open	
6B	2.75	C	Close	
13A	16.5	O	Open	
13A	21.25	C	Close	
WTP1-10	4.75	C	Close	
WTP2-7	20	C	Close	
WTP2-6	20	O	Open	

Base Model Changes for 4-3-01

Pipe	Time			Comment
12389	2	O	Open	
12389	7.25	C	Close	
12389	23.75	O	Open	
12317	1.25	O	Open	
12317	5	C	Close	
12375	0	O	Open	
12375	3	C	Close	
12421	0	O	Open	
12421	6	C	Close	

Completed on 3-13-04

Node	Time			Comment
WTP2-6	1	O	Open	Start Pump
WTP2-7	0	O	Open	Running all day
WTP1-10	0	O	Open	
WTP1-10	8.5	C	Close	Pump Shutdown
WTP1-10	23	O	Open	Start Pump
WTP1-11	0	O	Open	
WTP1-11	8.4	C	Close	Pump Shutdown
WTP1-11	9	O	Open	
WTP1-11	10	C	Close	
WTP1-11	11	O	Open	
WTP1-11	12	C	Close	
WTP1-11	13.6	O	Open	
WTP1-14	0	O	Open	
WTP1-14	8.3	C	Close	
WTP1-14	9	O	Open	
WTP1-14	10	C	Close	
WTP1-14	13	O	Open	
14A Booster	9.25	O	Open	
14A Booster	15.16	C	Close	
14A Booster	20.5	O	Open	
14A Booster	21.67	C	Close	
15 Booster	10	O	Open	
15 Booster	16	C	Close	
15 Booster	18.33	O	Open	
15 Booster	20.5	C	Close	
15 Booster	22.5	O	Open	
15 Booster	23	C	Close	
6A	8.33	O	Open	
6A	13	C	Close	
6C	8.5	O	Open	
6C	9	C	Close	
6C	10	O	Open	
6C	13	C	Close	
6C	21.67	O	Open	
6C	23	C	Close	
13A	7.16	O	Open	
13A	9.67	C	Close	
13A	10	O	Open	
13A	13.67	C	Close	
13B	19.25	O	Open	Set speed at 0.81
13B	22.25	C	Close	

Base Demand Factor Changes

Time	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
4/3/2001	0.52	0.48	0.50	0.51	0.44	0.49	1.01	1.12	1.19	0.96	0.73	0.85	0.84	0.78	0.80	0.75	0.94	0.96	0.89	0.97	1.07	0.97	0.80	0.59
7/4/2001	0.86	1.12	0.49	0.54	0.81	0.81	0.82	0.86	0.99	1.07	1.35	1.30	1.29	1.07	1.07	1.03	1.13	1.06	1.04	0.98	0.91	0.94	0.94	0.91
10/18/2002	0.69	0.58	0.59	0.60	0.64	0.90	1.00	1.32	0.91	0.99	1.15	1.08	1.04	1.11	1.11	0.96	1.06	0.98	1.01	1.09	0.92	0.91	0.81	0.79
10/15/2002	0.96	0.96	0.96	0.96	0.95	1.08	0.72	0.76	0.82	0.93	0.97	0.94	0.99	0.99	0.98	1.07	1.04	0.74	0.70	0.91	0.82	0.85	0.87	1.12

Initial Tank Levels

	Site #											
	14 (FT)	**14a** (FT)	**6** (FT)	**15** (FT)	**TAT** (FT)	**4** (FT)	**16** (FT)	**17** (FT)	**MF** (FT)	**SDL** (FT)	**13** (FT)	**BHTL** (FT)
Range	35	30	36.5	35	37.5	35	35	90	39.5	72	37.5	40
Overflow (MSL)	1002	1117	979.5	1107	1185.25	1000.5	1025.5	1115	1130	992	1022.5	1150
Bottom (MSL)	967	1087	943	1072	1147.75	965.5	990.5	1025	1090.5	920	985	1110
Oct 18/02 0:04:04	13.15	11.24	16.66	15.78	16.15	13.42	15.11	82.36	30.26	-	17.64	28.70
	980.15	1,098.24	959.66	1,087.78	1,163.90	978.92	1,005.61	1,107.36	1,120.76	**OTS**	1,002.64	1,138.70
Jul 04/01 0:01:09	30.31	8.98	30.02	13.53	-	11.02	12.85	65.30	35.26	-	13.48	32.03
	997.31	1,095.98	973.02	1,085.53	1,147.75	976.52	1,003.35	1,090.30	1,125.76	**OTS**	998.48	1,142.03
Apr 03/01 0:01:48	9.20	-	9.13	18.87	10.00	18.43	5.33	82.51	29.59	-	9.80	22.54
	976.20	**OTS**	952.13	1,090.87	1,157.75	983.93	**OTS**	1,107.51	1,120.09	**OTS**	994.80	1,132.54

Pump Inertia Computations for 6a

Device Data	
CV Time	0.01
CV Res	0
File (1-8)	1
Rated Hd	220
Rated Flw	4200
Rated Spd	1780
Inertia	139

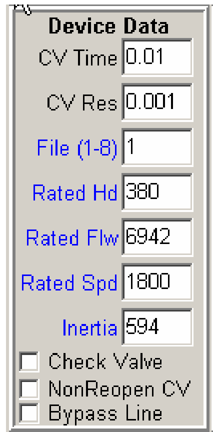
- [ ] Check Valve
- [ ] NonReopen CV
- [ ] Bypass Line

Pump Inertia Computations for WTP1-10 & 11

Device Data	
CV Time	0
CV Res	0
File (1-8)	1
Rated Hd	380
Rated Flw	5555
Rated Spd	1800
Inertia	400

- [ ] Check Valve
- [ ] NonReopen CV
- [ ] Bypass Line

Pump Inertia Computations for WTP1-14



APPENDIX C

CELERITY COMPUTATIONS

This Appendix includes calculation and spreadsheets created to compute and assign celerity (wave speed) values to each of the pipes in the surge models. Presented below are spreadsheets indicating how the different values of celerity were computed based on pipe size and material type. The values are based on no air entrainment.

Ductile Iron Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for DIP, wall thickness based on pressure class 350 pipe

English Units					D		e	K	E	ρ	μ		a	a	Rounded a
Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	Dia. (m)	Thick. (mm)	Thick (m)	Bulk Mod. (GN/m2)	Young M (GN/m2)	density (kg/m3)	Poisson's Ratio	c1	wave speed (m/s)	wave speed (ft/s)	wave speed (ft/s)
4	0.25	DI	water	102	0.10	6.35	0.0064	2.19E+09	1.72E+11	998	0.25	0.9375	1357	4453	4500
6	0.25	DI	water	152	0.15	6.35	0.0064	2.19E+09	1.72E+11	998	0.25	0.9375	1306	4285	4300
8	0.25	DI	water	203	0.20	6.35	0.0064	2.19E+09	1.72E+11	998	0.25	0.9375	1260	4134	4100
10	0.26	DI	water	254	0.25	6.60	0.0066	2.19E+09	1.72E+11	998	0.25	0.9375	1226	4023	4000
12	0.28	DI	water	305	0.30	7.11	0.0071	2.19E+09	1.72E+11	998	0.25	0.9375	1205	3953	4000
16	0.34	DI	water	406	0.41	8.64	0.0086	2.19E+09	1.72E+11	998	0.25	0.9375	1185	3889	3900
20	0.38	DI	water	508	0.51	9.65	0.0097	2.19E+09	1.72E+11	998	0.25	0.9375	1161	3809	3800
24	0.43	DI	water	610	0.61	10.92	0.0109	2.19E+09	1.72E+11	998	0.25	0.9375	1148	3765	3800
30	0.49	DI	water	762	0.76	12.45	0.0124	2.19E+09	1.72E+11	998	0.25	0.9375	1126	3694	3700
36	0.56	DI	water	914	0.91	14.22	0.0142	2.19E+09	1.72E+11	998	0.25	0.9375	1114	3656	3700

Cast Iron Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for CI, wall thickness based on pressure class 350 pipe

English Units					D		e	K	E	ρ	μ		a	a	Rounded a
Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	Dia. (m)	Thick. (mm)	Thick (m)	Bulk Mod. (GN/m2)	Young M (GN/m2)	density (kg/m3)	Poisson's Ratio	c1	wave speed (m/s)	wave speed (ft/s)	wave speed (ft/s)
4	0.35	CI	water	102	0.10	8.89	0.0089	2.19E+09	1.25E+11	998	0.25	0.9375	1359	4459	4500
6	0.38	CI	water	152	0.15	9.65	0.0097	2.19E+09	1.25E+11	998	0.25	0.9375	1320	4331	4300
8	0.41	CI	water	203	0.20	10.41	0.0104	2.19E+09	1.25E+11	998	0.25	0.9375	1289	4229	4200
10	0.44	CI	water	254	0.25	11.18	0.0112	2.19E+09	1.25E+11	998	0.25	0.9375	1264	4147	4100
12	0.48	CI	water	305	0.30	12.19	0.0122	2.19E+09	1.25E+11	998	0.25	0.9375	1247	4092	4100
16	0.5	CI	water	406	0.41	12.70	0.0127	2.19E+09	1.25E+11	998	0.25	0.9375	1199	3935	3900
20	0.57	CI	water	508	0.51	14.48	0.0145	2.19E+09	1.25E+11	998	0.25	0.9375	1180	3871	3900
24	0.63	CI	water	610	0.61	16.00	0.0160	2.19E+09	1.25E+11	998	0.25	0.9375	1162	3812	3800
30	0.73	CI	water	762	0.76	18.54	0.0185	2.19E+09	1.25E+11	998	0.25	0.9375	1145	3755	3800
36	0.81	CI	water	914	0.91	20.57	0.0206	2.19E+09	1.25E+11	998	0.25	0.9375	1126	3695	3700

PVC Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for PVC, wall thickness based on pressure class 200 pipe

English Units Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m2)	E Young M (GN/m2)	ρ density (kg/m3)	μ Poisson's Ratio	c1	a wave speed (m/s)	a wave speed (ft/s)	Rounded a wave speed (ft/s)
4	0.19	PVC	water	102	0.10	4.84	0.0048	2.19E+09	3.30E+09	998	0.25	0.9375	395	1296	1300
6	0.33	PVC	water	152	0.15	8.47	0.0085	2.19E+09	3.30E+09	998	0.25	0.9375	424	1391	1400
8	0.44	PVC	water	203	0.20	11.29	0.0113	2.19E+09	3.30E+09	998	0.25	0.9375	424	1391	1400
10	0.56	PVC	water	254	0.25	14.11	0.0141	2.19E+09	3.30E+09	998	0.25	0.9375	424	1391	1400
12	0.67	PVC	water	305	0.30	16.93	0.0169	2.19E+09	3.30E+09	998	0.25	0.9375	424	1391	1400

HDPE Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for HDPE, wall thickness based on pressure class 200 pipe

English Units Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m2)	E Young M (GN/m2)	ρ density (kg/m3)	μ Poisson's Ratio	c1	a wave speed (m/s)	a wave speed (ft/s)	Rounded a wave speed (ft/s)
14	1.56	HDPE	water	356	0.36	39.51	0.0395	2.19E+09	9.49E+08	998	0.25	0.9375	327	1074	1100

AC Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for AC, wall thickness based on pressure class 200 pipe

English Units Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m2)	E Young M (GN/m2)	ρ density (kg/m3)	μ Poisson's Ratio	c1	a wave speed (m/s)	a wave speed (ft/s)	Rounded a wave speed (ft/s)
4	0.46	AC	water	102	0.10	11.68	0.0117	2.19E+09	2.40E+10	998	0.25	0.9375	1122	3680	3700
6	0.66	AC	water	152	0.15	16.76	0.0168	2.19E+09	2.40E+10	998	0.25	0.9375	1111	3645	3600
8	0.76	AC	water	203	0.20	19.30	0.0193	2.19E+09	2.40E+10	998	0.25	0.9375	1075	3525	3500
10	0.96	AC	water	254	0.25	24.38	0.0244	2.19E+09	2.40E+10	998	0.25	0.9375	1077	3534	3500
12	1.09	AC	water	305	0.30	27.69	0.0277	2.19E+09	2.40E+10	998	0.25	0.9375	1063	3488	3500
16	1.36	AC	water	406	0.41	34.54	0.0345	2.19E+09	2.40E+10	998	0.25	0.9375	1046	3431	3400
20	1.82	AC	water	508	0.51	46.23	0.0462	2.19E+09	2.40E+10	998	0.25	0.9375	1064	3489	3500

Lock Joint (LJ) Computation

Wave speed = a = ((K/ρ) / (1 + (KDc1)/(Ee))1/2
c1 = 1 - μ2 for pipe anchored upstream end only

Dimensions are nominal for LJ, wall thickness based on pressure class 250 pipe

English Units					D		e	K	E	ρ	μ		a	a	Rounded a
Dia (in)	Thick. (in)	Mat'l Type	Fluid Type	Dia. (mm)	Dia. (m)	Thick. (mm)	Thick (m)	Bulk Mod. (GN/m2)	Young M (GN/m2)	density (kg/m3)	Poisson's Ratio	c1	wave speed (m/s)	wave speed (ft/s)	wave speed (ft/s)
16	1.00	LJ	water	406	0.41	25.40	0.0254	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695	3700
20	1.25	LJ	water	508	0.51	31.75	0.0318	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695	3700
24	1.50	LJ	water	610	0.61	38.10	0.0381	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695	3700
30	1.88	LJ	water	762	0.76	47.63	0.0476	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695	3700
36	2.25	LJ	water	914	0.91	57.15	0.0572	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695	3700

Weighted Wave Speed Calculation

Total KAW Pipe

Diameter (in)	Material Type	Length (ft)	Wavespeed (ft/s)	Weighted Product	
3	AC	39,900	3,700	147,630,000	
4	AC	235,343	3,700	870,769,100	
6	AC	539,615	3,600	1,942,614,000	
8	AC	480,739	3,500	1,682,586,500	
10	AC	3,086	3,500	10,801,000	
12	AC	273,879	3,500	958,576,500	
16	AC	500	3,500	1,750,000	
1.25	C.I.	2,086			
2	C.I.	74,780			
2.25	C.I.	77,194			
3	C.I.	-			
4	C.I.	92,634	4,500	416,853,000	
6	C.I.	962,960	4,300	4,140,728,000	
8	C.I.	805,245	4,200	3,382,029,000	
10	C.I.	24,796	4,100	101,663,600	
12	C.I.	287,392	4,100	1,178,307,200	
16	C.I.	19,022	3,900	74,185,800	
20	C.I.	9,361	3,900	36,507,900	
16	CON.	54,283	3,700	200,847,100	
20	CON.	18,136	3,700	67,103,200	
24	CON.	83,387	3,700	308,531,900	
30	CON.	48,168	3,700	178,221,600	
4	D.I.	28,243	4,500	127,093,950	
6	D.I.	345,419	4,300	1,485,301,270	
8	D.I.	1,610,853	4,100	6,604,497,300	
10	D.I.	88	4,000	352,000	
12	D.I.	480,102	4,000	1,920,408,000	
16	D.I.	179,036	3,900	698,240,400	
20	D.I.	12,116	3,800	46,040,800	
24	D.I.	234,652	3,800	891,677,600	
30	D.I.	59,034	3,700	218,425,800	
36	D.I.	368	3,700	1,361,600	
3	GAL	767			
2	GAL.	14,828			
4	GAL.	1,213			
14	PEP	3,450	1,100	3,795,000	
1	PVC	11			
2	PVC	71,497			
2.5	PVC	43,160			
3	PVC	115,400			
4	PVC	39,605	1,300	51,486,500	
6	PVC	165,725	1,400	232,015,000	
8	PVC	361,758	1,400	506,461,200	
12	PVC	20,268	1,400	28,375,200	
3	STEEL	45			
4	STEEL	60			
		7,519,163		28,515,237,020	3792.3

Weighted Wave Speed all Pipe	3800
Weighted Wave Speed PVC	1400
Weighted Wave Speed DIP	4100
Weighted Wave Speed CON	3700
Weighted Wave Speed CI	4200
Weighted Wave Speed AC	3600

Effect of Air Entrainment

Wave speed = a = ((K/ρ) / (1 + (KDc$_1$)/(Ee))$^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air a' = a / ((1+C$_2$m/P$_a$^{2}))$^{1/2}$
c_2 = RTK / (1 + KD/Ee)

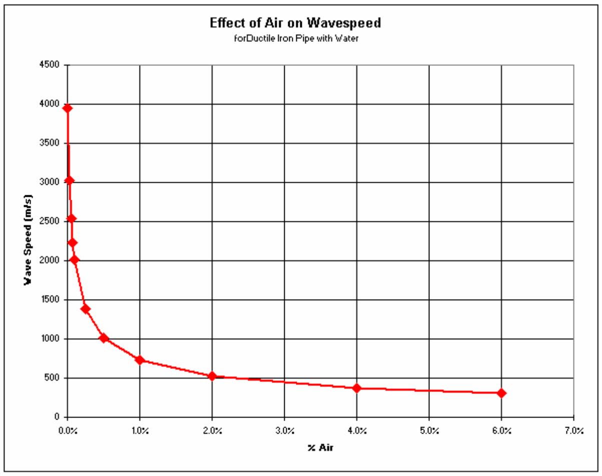
Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
Ductile	water	304.8	12	0.3048	7.11	0.007112	2.19E+09	1.72E+11	998	0.25	0.9375	1205	3953

R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
42.85714

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	1.191E+14	1204.9	3953
0.025%	0.001	1.191E+14	921.8	3024
0.050%	0.003	1.191E+14	775.0	2543
0.075%	0.004	1.191E+14	681.5	2236
0.100%	0.006	1.191E+14	615.4	2019
0.250%	0.015	1.191E+14	423.8	1390
0.500%	0.030	1.191E+14	309.4	1015
1.000%	0.059	1.191E+14	222.5	730
2.000%	0.119	1.191E+14	158.7	521
4.000%	0.238	1.191E+14	112.7	370
6.000%	0.357	1.191E+14	92.1	302



% Air	a' / a
0.000%	1.00
0.025%	0.77
0.050%	0.64
0.075%	0.57
0.100%	0.51
0.250%	0.35
0.500%	0.26
1.000%	0.18
2.000%	0.13
4.000%	0.09
6.000%	0.08

Wave speed = a = ((K/ρ) / (1 + (KDc$_1$)/(Ee))$^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air a' = a / ((1+C$_2$m/P$_a$^{2}))$^{1/2}$
c_2 = RTK / (1 + KD/Ee)

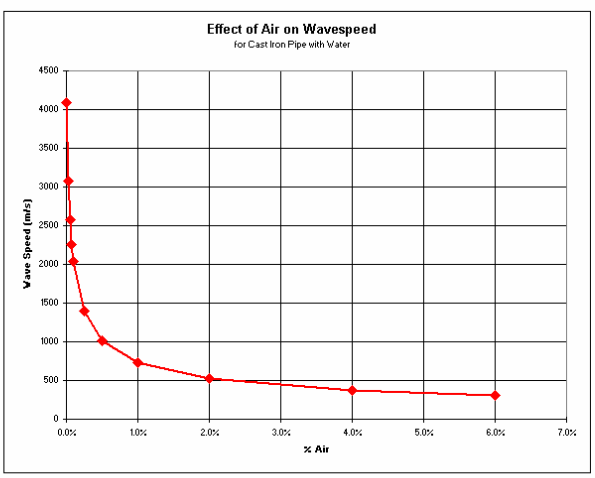
Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
Cast Iron	water	304.8	12	0.3048	12.19	0.012192	2.19E+09	1.25E+11	998	0.25	0.9375	1247	4092

R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
25

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	1.281E+14	1247.2	4092
0.025%	0.001	1.281E+14	939.8	3083
0.050%	0.003	1.281E+14	785.2	2576
0.075%	0.004	1.281E+14	688.2	2258
0.100%	0.006	1.281E+14	620.1	2034
0.250%	0.015	1.281E+14	424.9	1394
0.500%	0.030	1.281E+14	309.6	1016
1.000%	0.059	1.281E+14	222.4	730
2.000%	0.119	1.281E+14	158.5	520
4.000%	0.238	1.281E+14	112.5	369
6.000%	0.357	1.281E+14	92.0	302



% Air	a' / a
0.000%	1.00
0.025%	0.75
0.050%	0.63
0.075%	0.55
0.100%	0.50
0.250%	0.34
0.500%	0.25
1.000%	0.18
2.000%	0.13
4.000%	0.09
6.000%	0.07

Wave speed = a = ((K/ρ) / (1 + (KDc_1)/(Ee))$^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air $a' = a / ((1+C_2 m/P_a^2))^{1/2}$
$c_2 = RTK / (1 + KD/Ee)$

Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
PVC	water	304.8	12	0.3048	16.93	0.016933	2.19E+09	3.30E+09	998	0.25	0.9375	424	1391

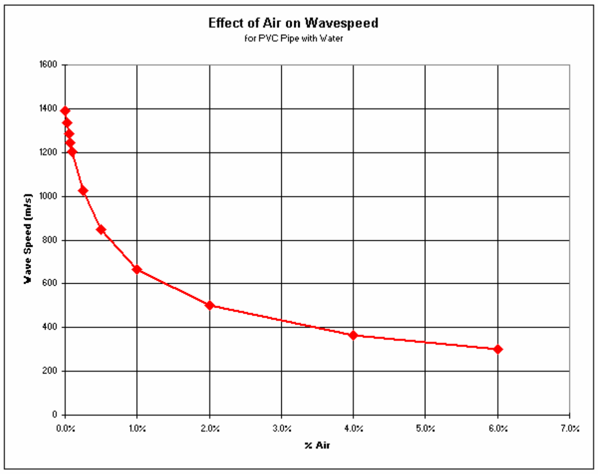
R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
18

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	1.423E+13	424.1	1391
0.025%	0.001	1.423E+13	407.3	1336
0.050%	0.003	1.423E+13	392.2	1287
0.075%	0.004	1.423E+13	378.8	1243
0.100%	0.006	1.423E+13	366.6	1203
0.250%	0.015	1.423E+13	312.2	1024
0.500%	0.030	1.423E+13	258.5	848
1.000%	0.059	1.423E+13	202.6	665
2.000%	0.119	1.423E+13	152.2	499
4.000%	0.238	1.423E+13	111.3	365
6.000%	0.357	1.423E+13	91.9	301

% Air	a' / a
0.000%	1.00
0.025%	0.96
0.050%	0.92
0.075%	0.89
0.100%	0.86
0.250%	0.74
0.500%	0.61
1.000%	0.48
2.000%	0.36
4.000%	0.26
6.000%	0.22



Wave speed = a = ((K/ρ) / (1 + (KDc_1)/(Ee))$^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air $a' = a / ((1+C_2 m/P_a^2))^{1/2}$
$c_2 = RTK / (1 + KD/Ee)$

Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
HDPE	water	304.8	12	0.3048	39.51	0.039511	2.19E+09	9.49E+08	998	0.25	0.9375	352	1155

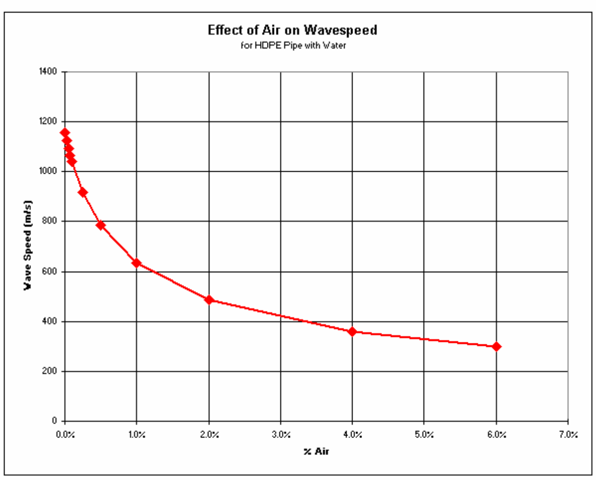
R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
7.714286

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	9.792E+12	352.2	1155
0.025%	0.001	9.792E+12	342.3	1123
0.050%	0.003	9.792E+12	333.3	1093
0.075%	0.004	9.792E+12	324.9	1066
0.100%	0.006	9.792E+12	317.2	1041
0.250%	0.015	9.792E+12	280.0	919
0.500%	0.030	9.792E+12	239.4	785
1.000%	0.059	9.792E+12	193.0	633
2.000%	0.119	9.792E+12	148.1	486
4.000%	0.238	9.792E+12	109.6	360
6.000%	0.357	9.792E+12	91.0	299

% Air	a' / a
0.000%	1.00
0.025%	0.97
0.050%	0.95
0.075%	0.92
0.100%	0.90
0.250%	0.79
0.500%	0.68
1.000%	0.55
2.000%	0.42
4.000%	0.31
6.000%	0.26



Wave speed = $a = \left((K/\rho) / (1 + (KDc_1)/(Ee)) \right)^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air $a' = a / \left((1+C_2 m/P_a^2) \right)^{1/2}$
$c_2 = RTK / (1 + KD/Ee)$

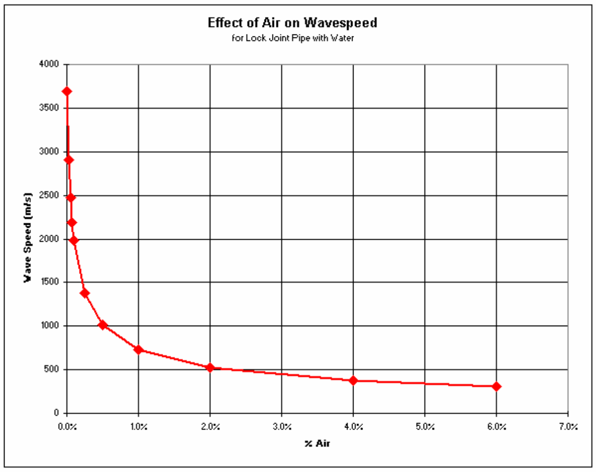
Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
LJ	water	406.4	16	0.4064	25.40	0.0254	2.19E+09	4.50E+10	998	0.25	0.9375	1126	3695

R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
16

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	1.035E+14	1126.2	3695
0.025%	0.001	1.035E+14	886.1	2907
0.050%	0.003	1.035E+14	754.0	2474
0.075%	0.004	1.035E+14	667.5	2190
0.100%	0.006	1.035E+14	605.3	1986
0.250%	0.015	1.035E+14	421.0	1381
0.500%	0.030	1.035E+14	308.7	1013
1.000%	0.059	1.035E+14	222.5	730
2.000%	0.119	1.035E+14	158.9	521
4.000%	0.238	1.035E+14	112.9	370
6.000%	0.357	1.035E+14	92.3	303



% Air	a' / a
0.000%	1.00
0.025%	0.79
0.050%	0.67
0.075%	0.59
0.100%	0.54
0.250%	0.37
0.500%	0.27
1.000%	0.20
2.000%	0.14
4.000%	0.10
6.000%	0.08

Wave speed = $a = \left((K/\rho) / (1 + (KDc_1)/(Ee)) \right)^{1/2}$
$c_1 = 1 - \mu^2$ for pipe anchored upstream end only
wave speed with air $a' = a / \left((1+C_2 m/P_a^2) \right)^{1/2}$
$c_2 = RTK / (1 + KD/Ee)$

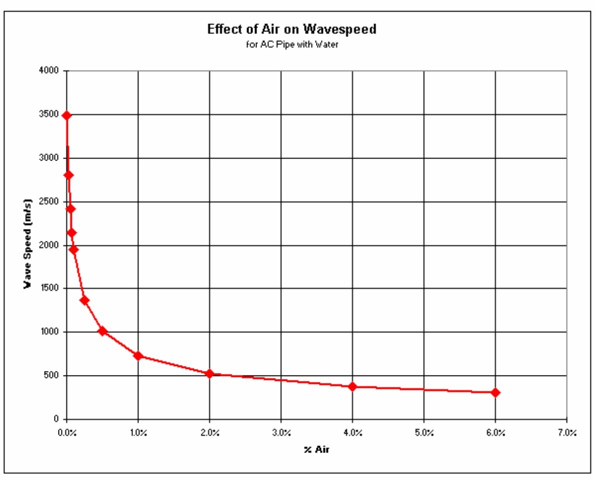
Material Type	Liquid Type	Dia. (mm)	Dia. (inch)	D Dia. (m)	Thick. (mm)	e Thick (m)	K Bulk Mod. (GN/m^2)	E Young M (GN/m^3)	ρ density (kg/m^3)	μ Poisson's Ratio	c_1	a wave speed (m/s)	a wave speed (ft/s)
AC	water	304.8	12	0.3048	27.69	0.027686	2.19E+09	2.40E+10	998	0.25	0.9375	1063	3488

R	287	J/Kg K
T	293	deg K
Pa	500000	Pa
	5	bar

D/e
11.00917

PV=mRT m= 5.95 kg/m^3

% Air	mass air (kg/m3)	c_2	a' wave speed (m/s)	a' wave speed (ft/s)
0.000%	0.000	9.187E+13	1063.1	3488
0.025%	0.001	9.187E+13	854.9	2805
0.050%	0.003	9.187E+13	734.9	2411
0.075%	0.004	9.187E+13	654.4	2147
0.100%	0.006	9.187E+13	595.7	1954
0.250%	0.015	9.187E+13	418.2	1372
0.500%	0.030	9.187E+13	307.8	1010
1.000%	0.059	9.187E+13	222.4	730
2.000%	0.119	9.187E+13	159.0	522
4.000%	0.238	9.187E+13	113.1	371
6.000%	0.357	9.187E+13	92.5	303



% Air	a' / a
0.000%	1.00
0.025%	0.80
0.050%	0.69
0.075%	0.62
0.100%	0.56
0.250%	0.39
0.500%	0.29
1.000%	0.21
2.000%	0.15
4.000%	0.11
6.000%	0.09

APPENDIX D

ABBREVIATIONS

The following is a list of abbreviations used throughout this work.

Abbreviation	Used for:
MGD	million gallons per day
cfs	cubic feet per second
gpm	gallons per minute
fps	feet per second
psi	pounds per square inch
VFD	variable frequency drive
SCADA	supervisory control and data acquisition
Hp or HP	horsepower

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VITA

Richard C. Svindland, P.E. was born in Beaumont, Texas in 1968. Mr. Svindland attended the International School of Geneva (Switzerland), graduated from Brandywine High School in Wilmington, Delaware in 1986 and earned a Bachelor of Civil Engineer degree from Georgia Institute of Technology in Atlanta, Georgia in 1990. In the same year he passed the Engineer-In-Training (EIT) examination and started his professional engineering career as an EIT with the Atlanta engineering firm, Wiedeman and Singleton, Inc. In 1993, Mr. Svindland joined another Atlanta firm, Keck & Wood, Inc. as a Project Engineer, EIT. In 1995, Mr. Svindland became a Registered Professional Engineer in the State of Georgia. In 1997, Mr. Svindland returned to Wiedeman and Singleton, Inc. as a Project Manager / Design Team Leader. In 1999, Mr. Svindland moved to Lexington, Kentucky and joined Kentucky American Water (KAW) as an Operations Engineer. In 2000, Mr. Svindland became a Registered Professional Engineer in the Commonwealth of Kentucky and in 2001 started graduate course work at the University of Kentucky. Mr. Svindland was promoted to Senior Operations Engineer with KAW in July of 2001 and was further promoted to Technical Services Manager Southeast Region of American Water (parent company of KAW) in July 2004. Mr. Svindland is an active member of the American Society of Civil Engineers (ASCE) and American Water Works Association (AWWA). In 2003, the Kentucky Section of ASCE named Mr. Svindland the Civil Engineer of the Year in Industry.